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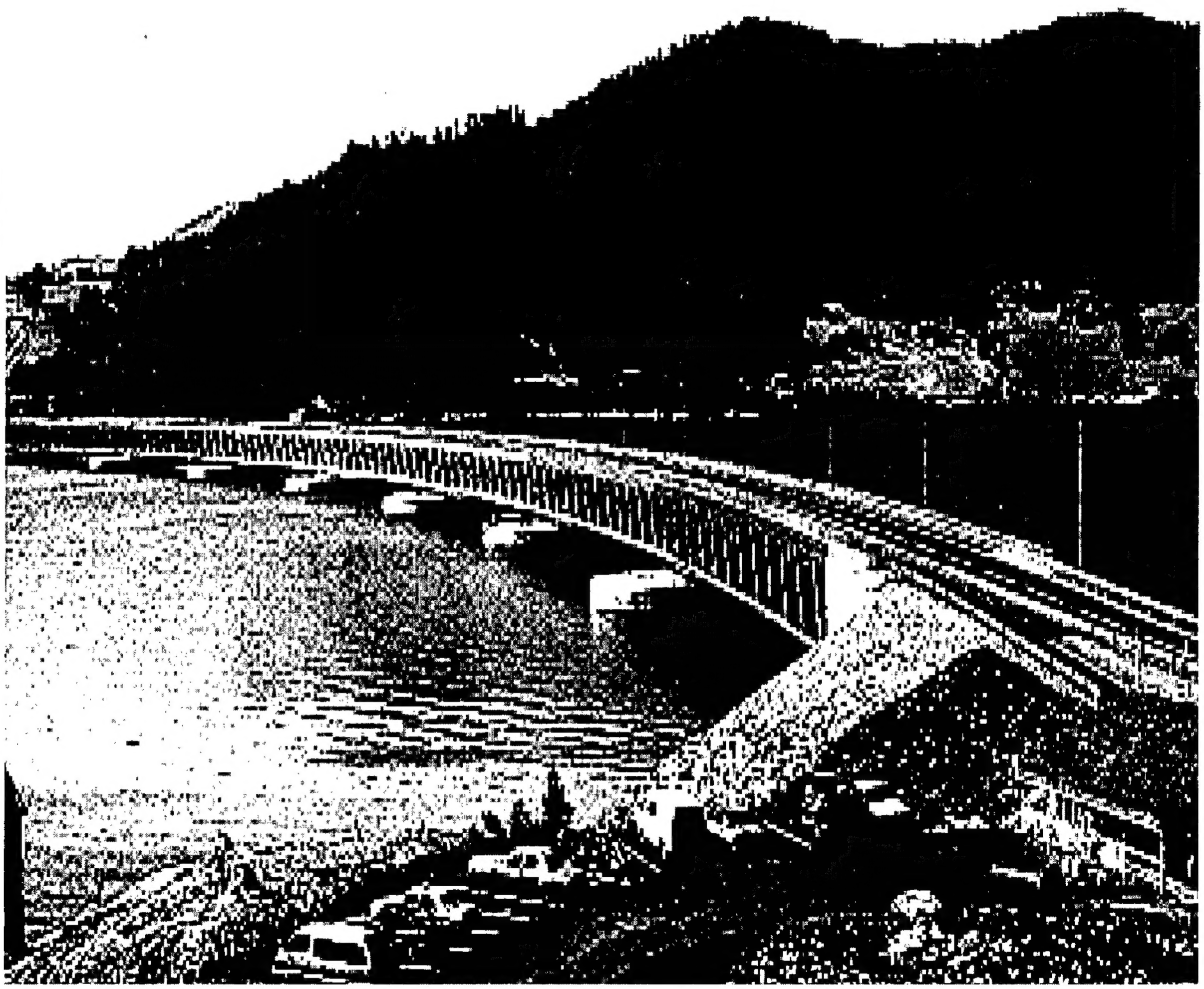
Innovations for Navigation Projects Research Program

Overview of Current Prestressing Technology in Offshore Structures

Sam X. Yao, Ben C. Gerwick, and Dale E. Berner

November 2000



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Overview of Current Prestressing Technology in Offshore Structures

by Sam X. Yao, Ben C. Gerwick, and Dale E. Berner

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Contents

Preface

The work described in this report was authorized by Headquarters, U.S. Army Corps of Engineers (HQUSACE), as part of the Innovations for Navigation Projects (INP) Research Program. The work was performed under Work Unit 33154, "Post-Tensioning or Prestressing for Hydraulic Structures," managed at the U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, MS. Dr. Robert L. Hall, ERDC Geotechnical and Structures Laboratory (GSL), was the Principal Investigator.

Dr. Tony C. Liu was the INP Coordinator at the Directorate of Research and Development, HQUSACE. Mr. William H. McAnally, ERDC Coastal and Hydraulics Laboratory, was the Technical Director of Navigation Programs, and Dr. Stanley C. Woodson, ERDC GSL, was the INP Program Manager.

This report was prepared by Drs. Sam X. Yao, Ben C. Gerwick, and Dale E. Berner of Ben C. Gerwick, Inc., under contract DACW39-99-P0577.

The work was monitored at ERDC GSL by Dr. Hall, under the general supervision of Dr. Mosher, Chief, Geosciences and Structures Division, and Dr. Michael J. O'Connor, Director, GSL.

At the time of publication of this report, Dr. James R. Houston was Director of ERDC, and COL James S. Weller, EN, was Commander.

1 Introduction

Background

In recent years, the U.S. Army Corps of Engineers has launched major developments in underwater construction of locks and dams, or the so-called "offsite prefabricated" or "in-the-wet" construction method. This innovative method uses precast concrete modules as the in situ form into which tremie concrete is placed directly without use of a cofferdam. In-depth studies have shown that the in-the-wet construction method will not only lead to substantial savings in cost and construction time, but will also alleviate the river traffic condition during construction, substantially reducing or eliminating the downtime of the navigation facilities during rehabilitation.

The in-the-wet construction method potentially entails a wide range of innovative marine construction methods, such as the float-in and lift-in methods for offsite prefabricated concrete components. Prestressing technology has been an essential part of offsite prefabrication and onsite installation of marine structures, such as offshore concrete platforms, floating concrete bridges, and marine terminals.

Throughout its development over the last 5 decades, prestressed concrete has shown substantial advantages for offshore structures in terms of structural configuration optimization, strength, fatigue resistance, durability, crack control, weight control, constructibility, and overall economy.

With advancements in innovative construction methods, it is perceived that prestressed concrete will potentially find numerous applications for in-the-wet construction of navigation structures, especially for the fabrication, installation, and joining of precast concrete modules. Therefore, an overall evaluation of the current engineering design practice of prestressed concrete offshore structures has become an important part of the Corps' innovative effort.

Objective

The objective of this report is to provide a general review of the current design practice in offshore prestressed concrete structures. This information will be integrated into design guidelines being developed for prestressed concrete navigation structures.

Scope

This report provides evaluations of three aspects of prestressing technology as follows:

a. Applications of prestressing technology in offshore structures, especially in offshore concrete gravity-based structures.

b. Current design criteria of offshore prestressed concrete structures, including the load criteria, ultimate limit state design criteria, fatigue limit state design criteria, and crack control design criteria.

a. Durability design requirements and past durability performance of offshore prestressed concrete structures.

2 Applications of Prestressing Technology in Offshore Structures

The use of prestressing technology in offshore structures began with harbor and coastal construction in the 1950s. Historically, prestressed concrete piles and sheet piles were the first structural components used in marine construction. Subsequently, with substantial improvement and wide availability of heavy lift and transportation equipment for heavy prefabricated concrete elements, precast/prestressed concrete panels have been widely used in the deck construction since the 1960s.

In pier and wharf structures, precast concrete deck panels are prestressed in one or two directions and designed as one- or two-way deck slabs or utility trenches (see Figure 1). In such applications, cast-in-place concrete is placed only in the closure pour between the precast concrete deck elements. The precast elements are then posttensioned together to ensure the integrity and continuity of the deck. Sometimes, precast concrete panels are also used as in situ form on which cast-in-place concrete overlay is placed. The precast panels are designed to act monolithically with the cast-in-place concrete overlay.

Concurrent with the prestressing applications in coastal structures, major advancements have been made in the design and construction of prestressed concrete floating structures. Historically, reinforced concrete ships and vessels were constructed during World War I and II when steel was in short supply. This experience proved that reinforced concrete ships are technically feasible. However, due to the lack of crack control and fatigue resistance, the concrete vessels demanded higher maintenance expenditures than similar steel ships. The development of prestressing technology offered a different approach to concrete shipbuilding. By precompressing the concrete hulls, potential cracks are minimized and durability is substantially improved. As a result, the size of the vessels can be greatly increased.

Subsequently, engineers developed innovative designs for various floating structures, such as offshore oil or petroleum gas production/storage stations (Figure 2), floating bridges (Figures 3 and 4), floating dry docks, and marine terminals. These floating structures fully exploit the efficiency of prestressed

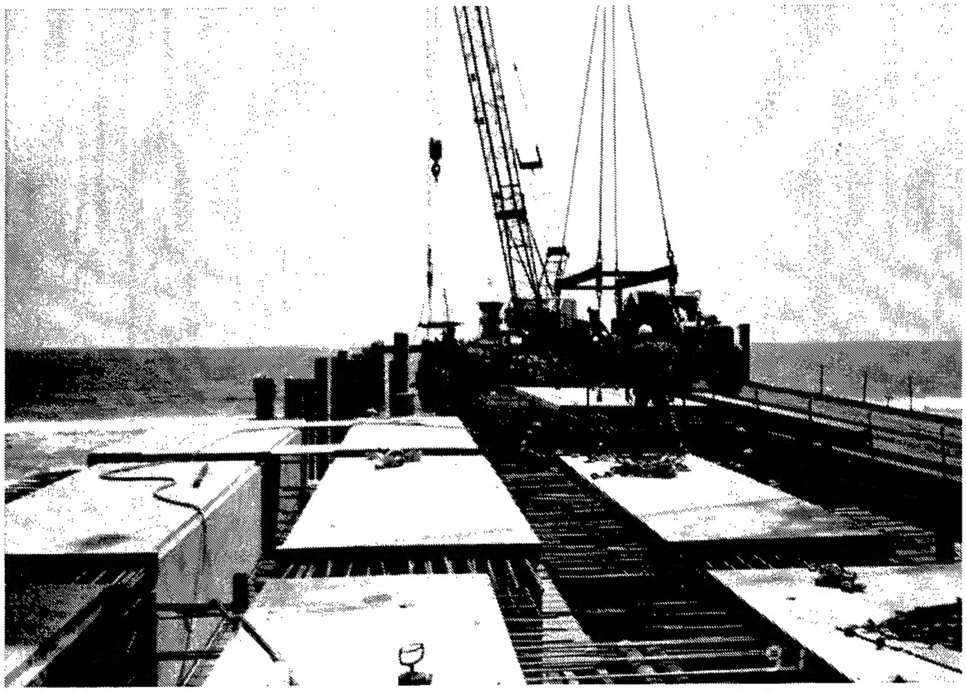


Figure 1. Pier deck construction with precast/prestressed concrete

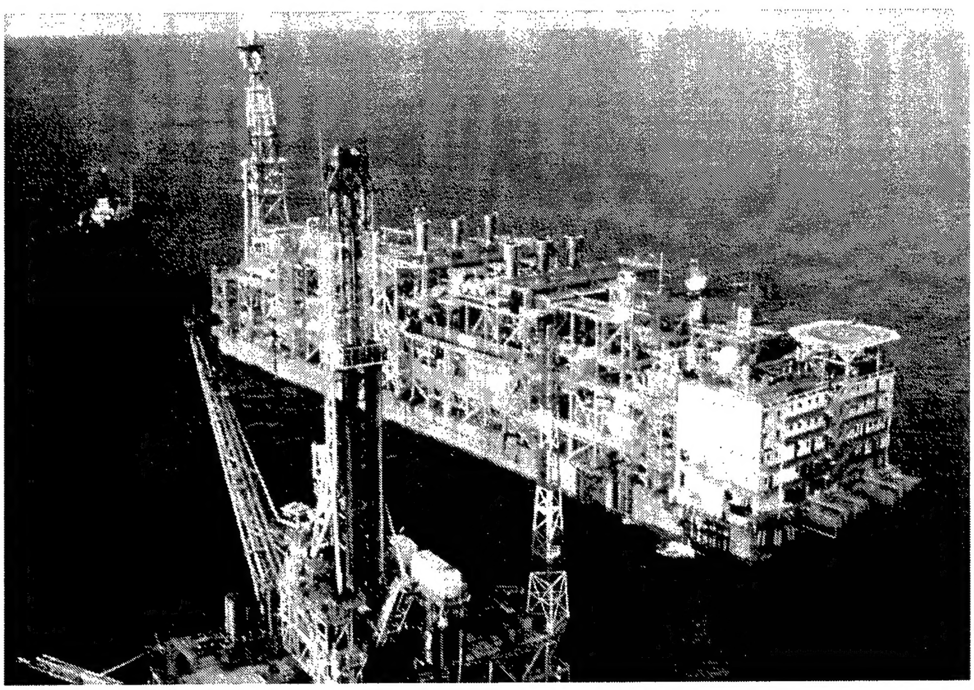
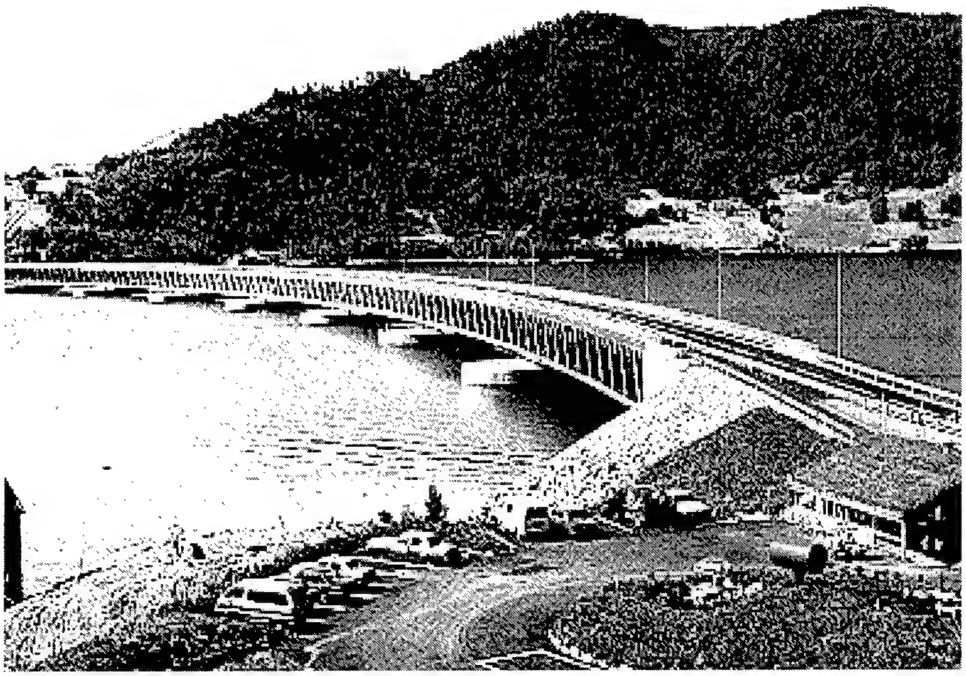
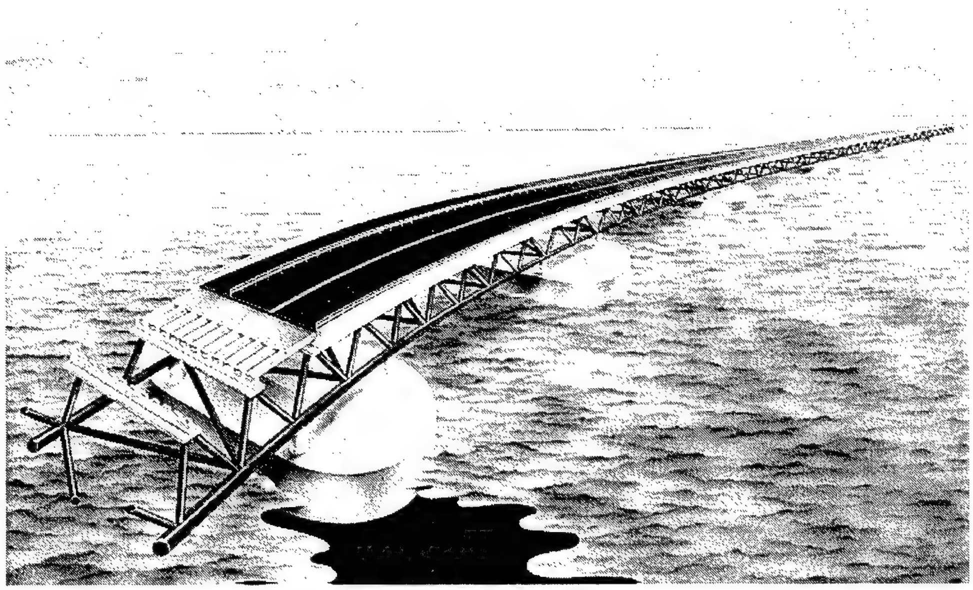


Figure 2. Nkossa prestressed concrete floating station for production and storage of oil/liquified petroleum gas, with its 30,000 tons of equipment (size: 220 m by 46 m by 16 m; completion: 1996)



a. Landside view of the bridge



b. Sectional view

Figure 3. Bergsoysund floating bridge in Norway, completed in 1992

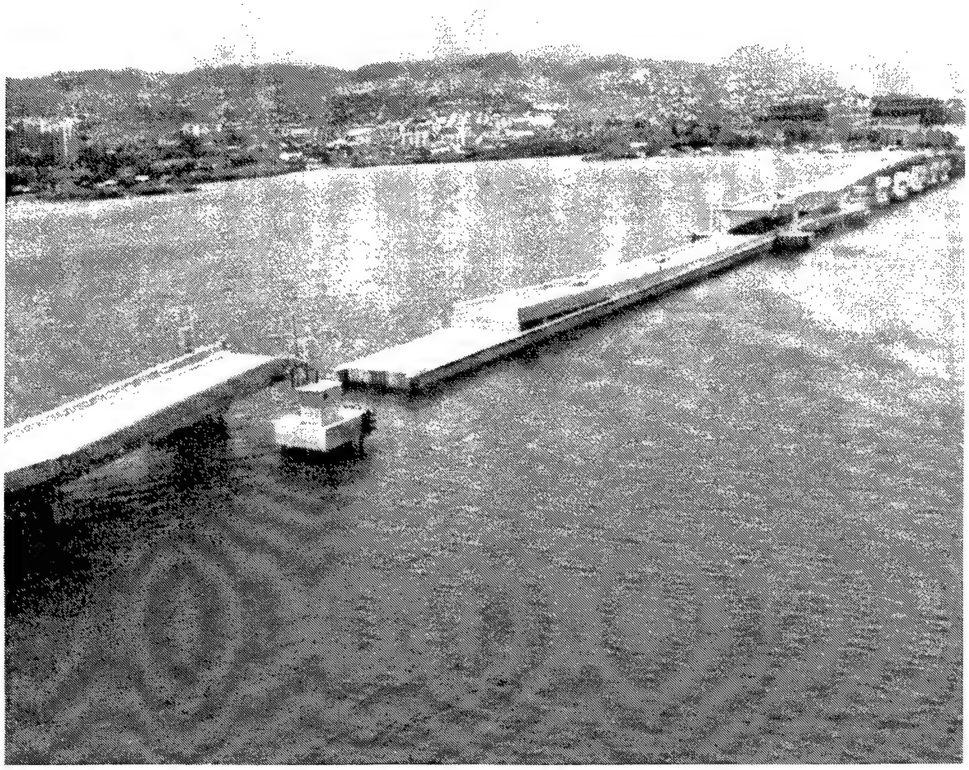


Figure 4. Admiral Clarey floating bridge, Pearl Harbor Naval Station, Hawaii

concrete in achieving crack control, watertightness, fatigue resistance, durability, abrasion resistance, structural strength, and redundancy. Prestressed concrete is especially adaptable to shell and double-curvature configurations that are very efficient for floating structures. Indeed, without prestressing, many of these marine structures would not be feasible. The performance of these prestressed concrete floating structures to date has been generally satisfactory.

The offshore application of prestressing technology reaches its pinnacle with the successful completion of many major gravity-based offshore platforms, such as Troll and Heidrun (Figures 5 and 6). These gigantic structures, displacing up to 800,000 tons and standing over 300 meters in the sea, are prestressed to withstand thousand cycles of strong waves, violent wind gusts and, in some cases, the impact of icebergs weighing millions of tons. A summary of the major offshore GBS platforms is provided in Table 1. Meanwhile, numerous small prestressed concrete platforms have been installed in relatively shallow depths of water in the Gulf of Mexico as drilling platforms, compressor stations and pump stations for gas production (Figure 7).

In the Arctic Ocean, prestressed concrete caissons such as Tarsiut and the Glomar Beaufort Sea One have withstood multiple years against ice flows containing pressure ridges in the Arctic environment (Figure 8). The Oosterschelde

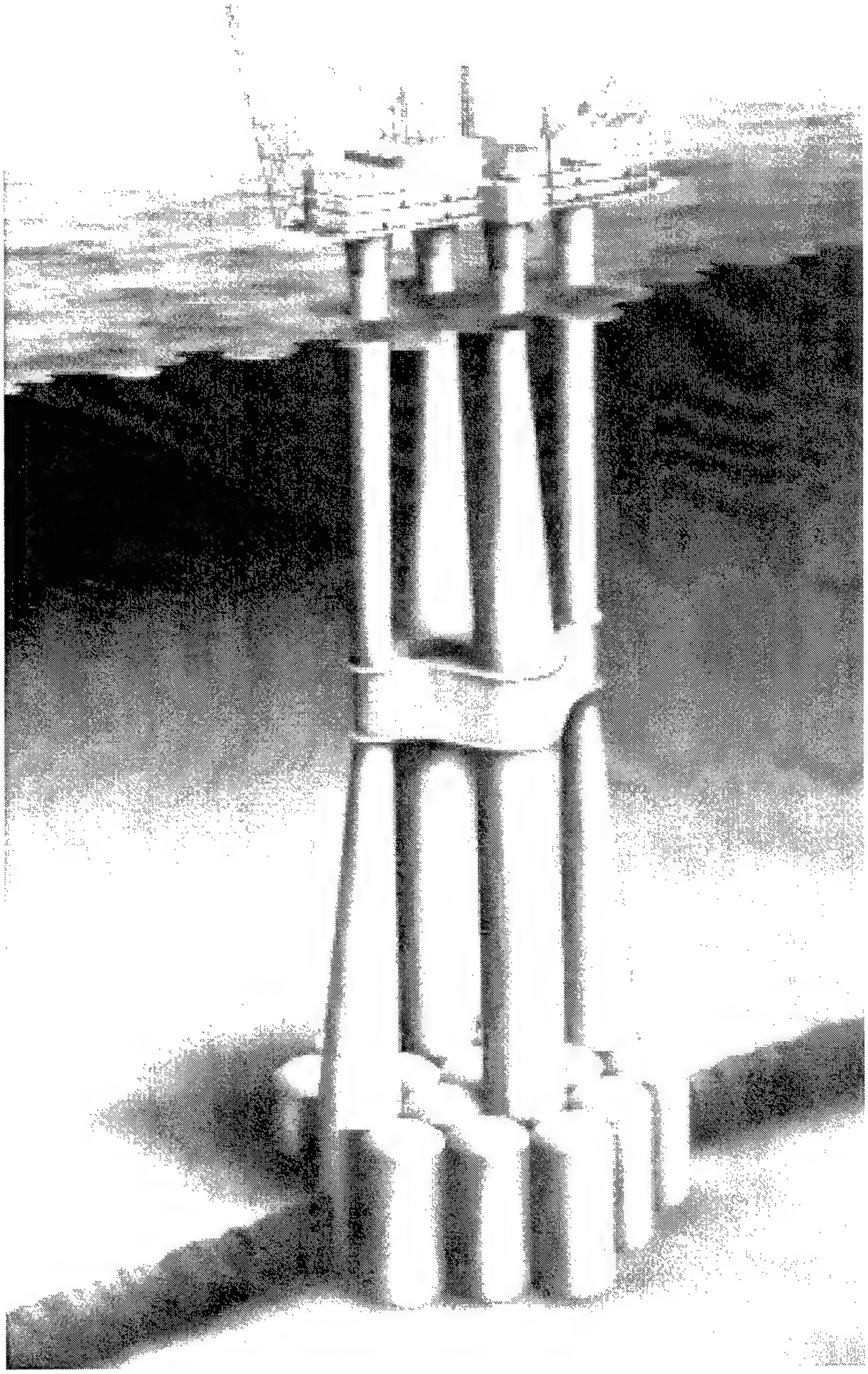


Figure 5. Troll Condeep in North Sea (water depth: 303 m; installation: 1995)

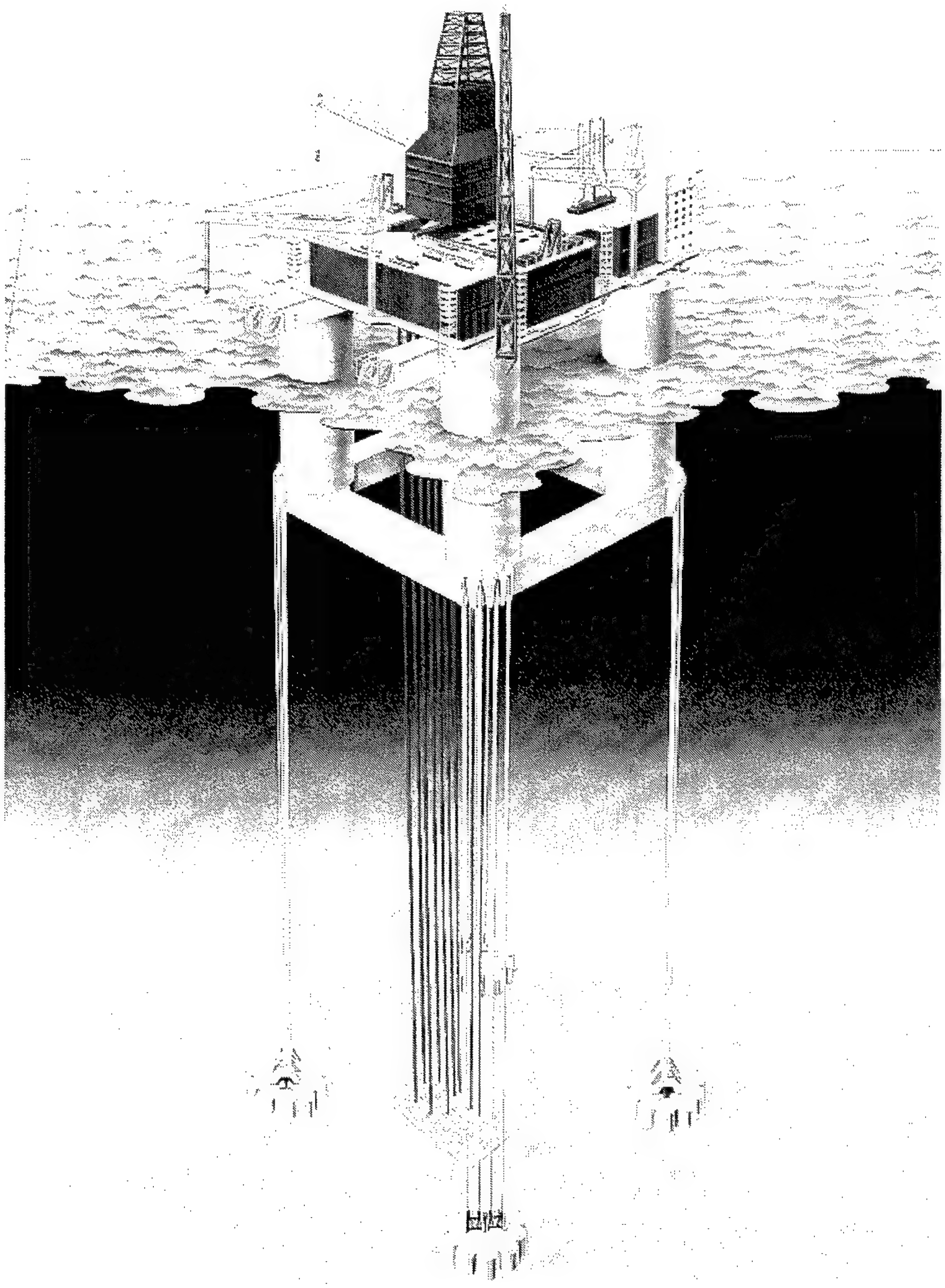
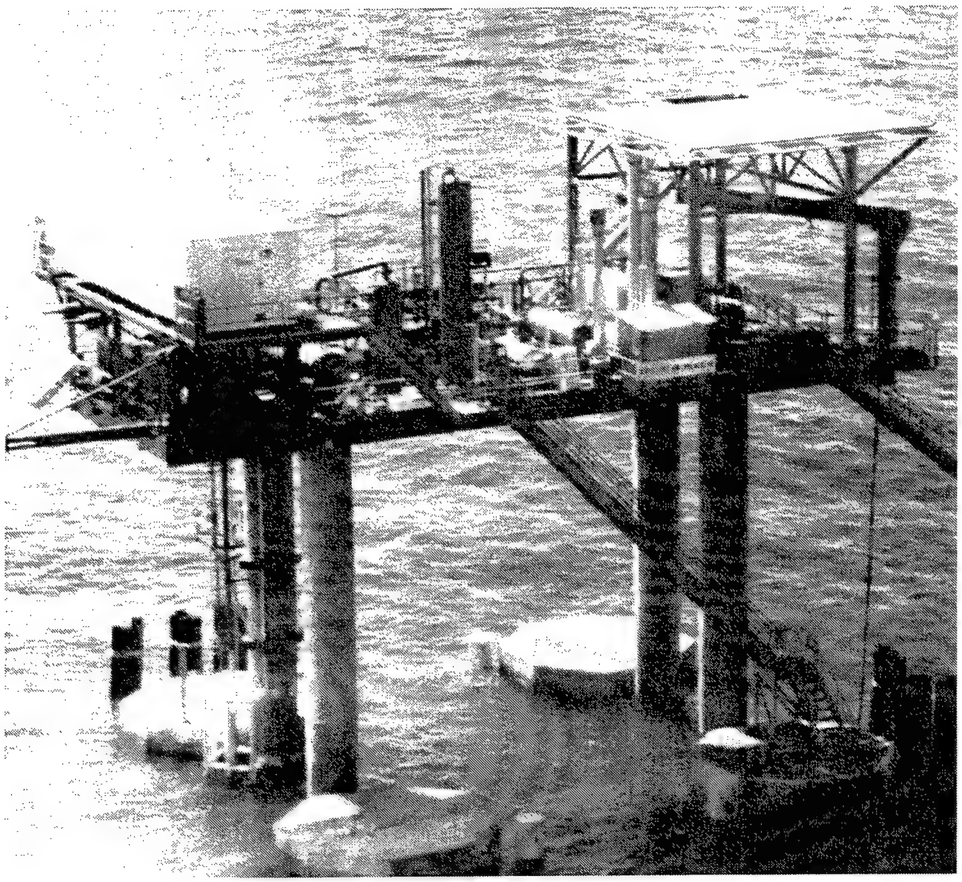


Figure 6. Heidrun tension leg platform in North Sea (water depth: 345 m; installation: 1995)

Table 1
Existing Offshore Concrete Gravity-Based Platforms

Platform Name	Operator	Water Depth (meter)	Concrete Volume (m^3)	Country of Fabrication	Year of Installation
Ekofisk 1	Philips	70	80,000	Norway	1973
Beryl A	Mobil	118	52,000	Norway	1975
Brent B	Shell	140	64,000	Norway	1975
Frigg CDP1	Elf	104	60,000	Norway	1975
Brent D	Shell	140	68,000	Norway	1976
Frigg TP1	Elf	104	49,000	U.K.	1976
Frigg MP2	Elf	94	60,000	Sweden	1976
Dunlin A	Shell	153	90,000	Holland	1977
Frigg TCP2	Elf	104	50,000	Norway	1977
Statfjord A	Mobil	145	87,000	Norway	1977
Cormorant A	Shell	149	120,000	U.K.	1978
Ninian Ctr.	Chevron	136	140,000	U.K.	1978
Brent C	Shell	141	105,000	U.K.	1978
Statfjord B	Mobil	145	140,000	Norway	1981
Statfjord C	Mobil	145	130,000	Norway	1984
Gullfaks A	Statoil	135	125,000	Norway	1986
Gullfaks B	Statoil	141	100,000	Norway	1987
Oseberg A	NorskHydro	109	120,000	Norway	1988
Gullfaks C	Statoil	216	240,000	Norway	1989
Ekofisk Protective Barrier	Philips	75	105,000	Norway	1989
Ravenspurn North	Hamilton	42	10,000	U.K.	1989
F3	NAM	43	21,000	Holland	1992
Sleipner A	Statoil	82	79,000	Norway	1993
Draugen	Shell	251	85,000	Norway	1993
Troll-GBS	Shell	303	221,000	Norway	1995
West Tuna	Esso	61	26,800	Australia	1995
Bream B	Esso	61	12,000	Australia	1995
Harding – TPG 500	British Petroleum	106	37,000	U.K.	1995
Hibernia	HMDC	80	166,000	Canada	1997



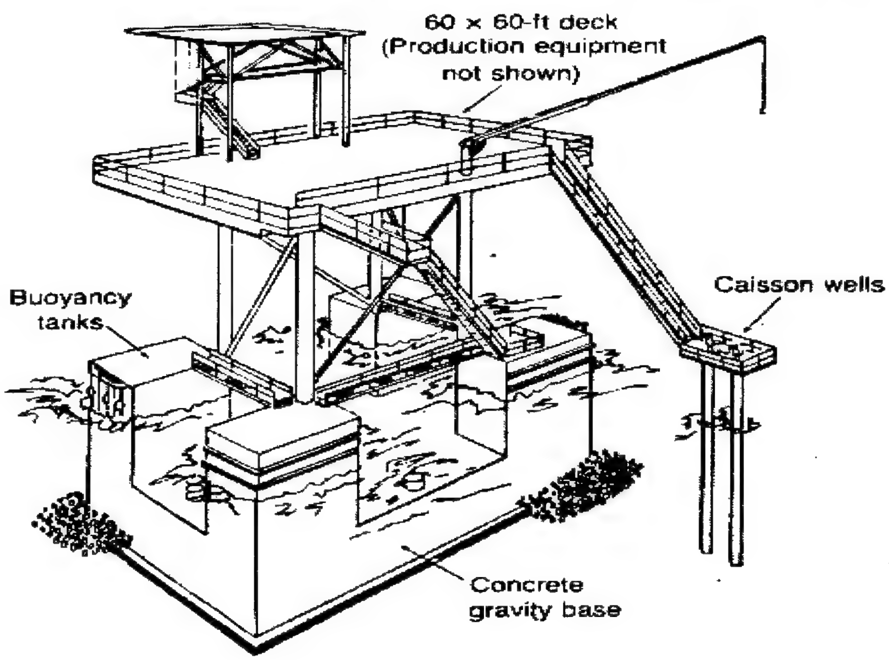
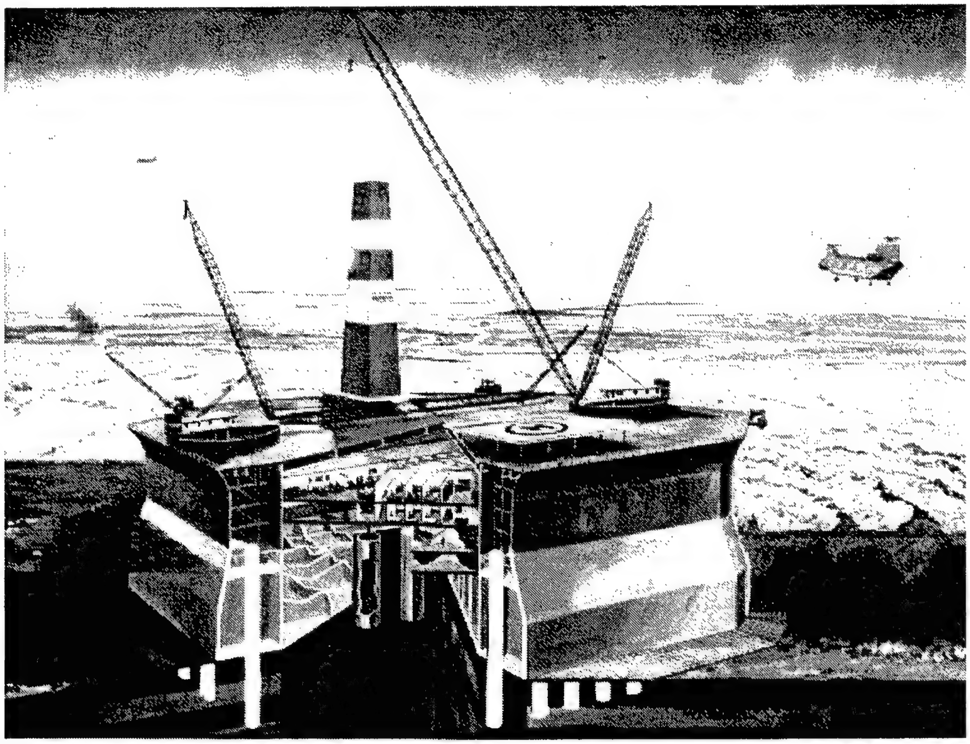
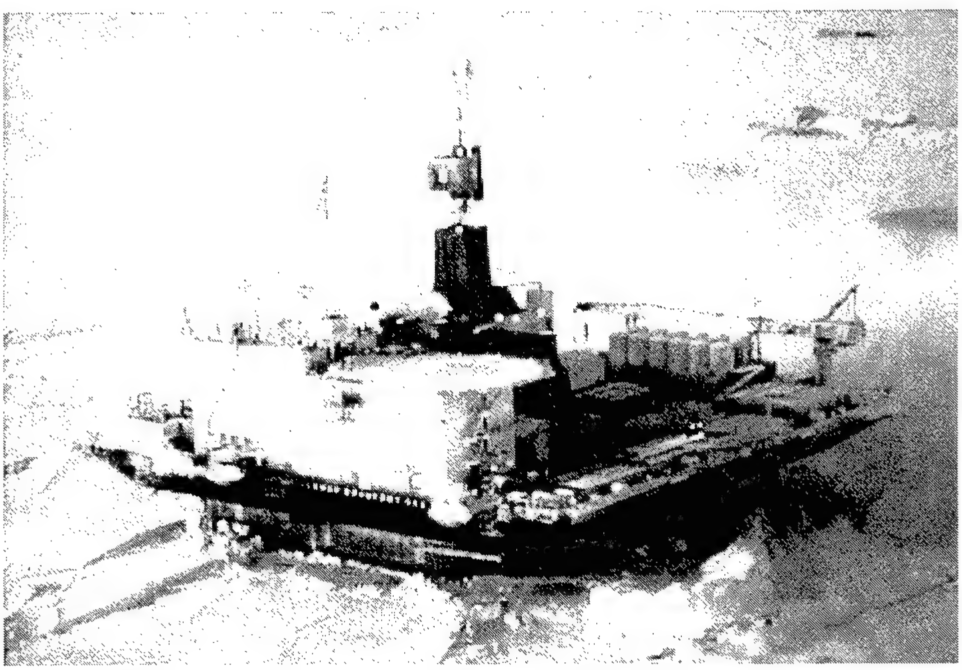


Figure 7. Oil production system in shallow water of the Gulf of Mexico



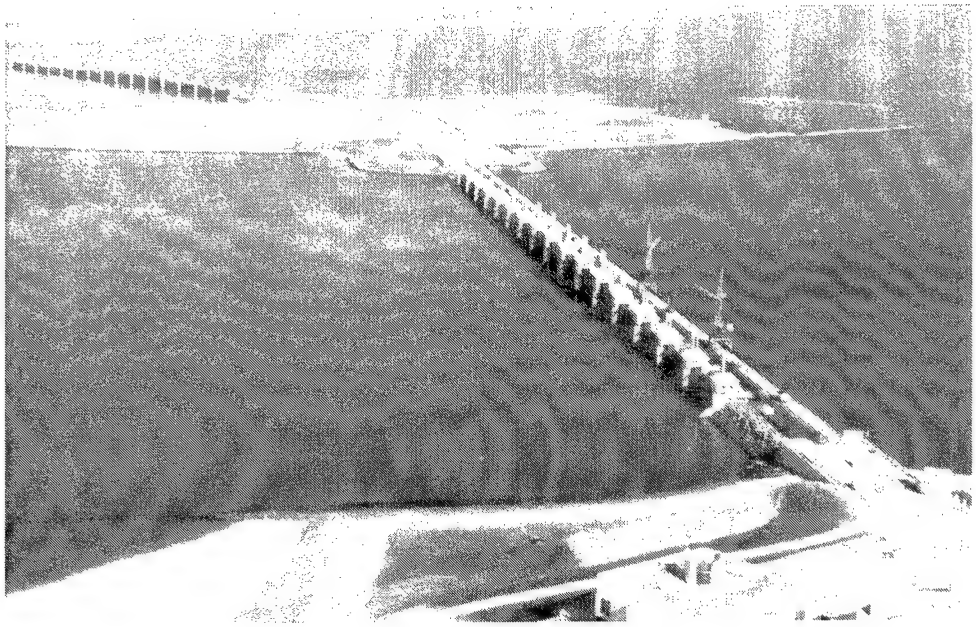
a. Sectional view



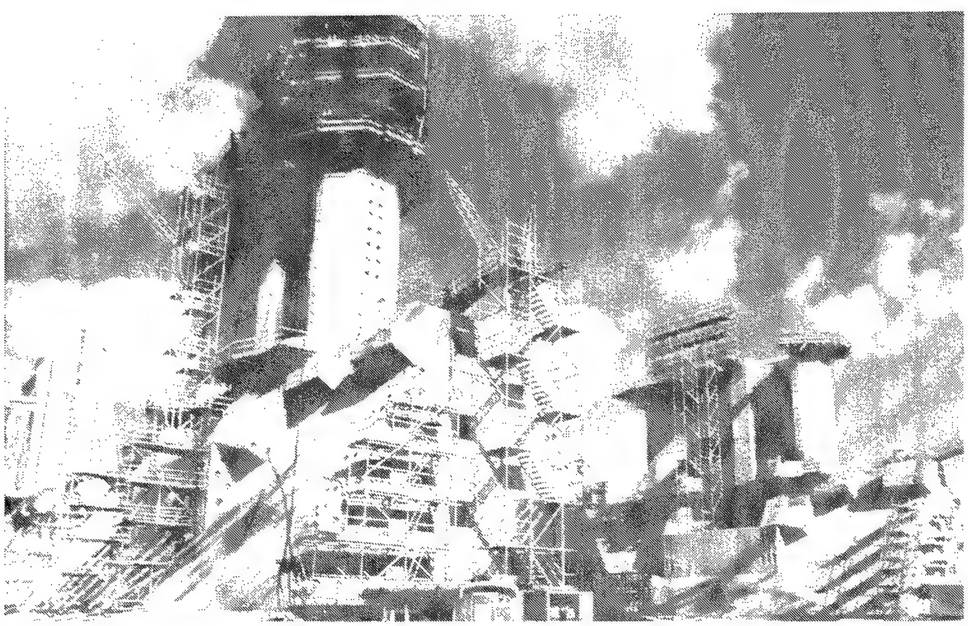
b. Super concrete island drilling system

Figure 8. Prestressed concrete caissons in the Arctic Ocean

Storm Surge Barrier, across the Eastern Scheldt Estuary of the Netherlands, is one of the great engineering achievements in 20th century. It consists of a series of gravity-based piers and hydraulically operated steel gates. The piers are 50 by 36 by 25 m precast concrete segments fully prestressed in three dimensions (Figure 9).



a. Landscape view



b. Construction of the prestressed concrete piers in a drydock

Figure 9. Oosterschelde storm surge barrier of the Netherlands

The primary reasons for the rapid acceptance of prestressing technology in the offshore applications are its constructibility, durability and economy. Marine construction entails overwater construction with various logistical implications. First, transportation of materials and provision of temporary support over water for construction are difficult and expensive. For example, the cost of formwork for cast-in-place concreting of a pier is typically on the order of 10 to 15 percent of the total construction cost. Second, access of workmen to the job site is difficult and time-consuming. The labor cost for marine work runs twice as high as that in precast concrete yards. Furthermore, onsite weather conditions are often unfavorable to large-scale concrete construction for a long period of time. These factors make it very difficult to construct high-quality cast-in-place concrete overwater.

For the same reasons, use of precast concrete segments can significantly improve the construction quality and achieve substantial savings through reduction in onsite construction time and costs. Unexpected construction downtime due to weather condition is minimized. Furthermore, standardized fabrication of precast concrete elements in an industrial manufacture process can significantly improve quality of the concrete structures. All these factors of marine construction emphasize the value of prefabrication and modular construction.

Prestressed concrete, if properly designed and fabricated, has proven to enhance the strength and serviceability of marine structures. Prestressing is especially effective in resisting shear, bending, and fatigue loads and in providing crack control and watertightness. These beneficial effects of prestressing are further discussed in Chapter 3.

3 Durability and Past Performance of Offshore Concrete Structures

Durability Design Requirements

Past experience with offshore concrete structures indicated that many different factors affect concrete durability performance in the marine environment. However, the single most important factor underlying the durability performance is permeability of the concrete. Serious deterioration of concrete structures starts only after aggressive materials in the surrounding environment are allowed to penetrate into the interior of concrete. Typical causes of permeable concrete include poorly proportioned concrete mixtures, insufficient concrete cover, bad construction joints, inadequate consolidation and curing, absence of proper air entrainment in freeze-thaw environments, serious cracking in concrete due to overloading, thermal stress, and dry shrinkage. Of critical importance to concrete durability are not only the concrete materials, but also the concrete production process (placement, consolidation, curing) and adequate structural design.

As the offshore construction industry is going through the learning curve over the years, so-called "ten golden rules for durable concrete in the sea" have been gradually established in a trial-and-error process. The key to durable offshore concrete structures has been found to be stringent requirements and careful control of design and construction, in conformance with the ten golden rules as described below.

a. Select high-quality materials.

(1) Cement: moderate C_3A (5 to 8 percent), moderate alkali content.

(2) Aggregates: check soundness and impurities, uniform grading, control content of fines to ensure slump retention.

(3) Admixture: select efficient water-reducing admixture, melamine-based high-range water-reducing (HRWR) admixture gives less set retardation at high dosages; check air spacing factor and specific surface frequently for frost resistance.

b. Get the mixture proportions right.

(1) Water-cement ratio (w/c) <0.45 (0.40 in splash zone) is imperative.

(2) Relatively high cement content (>356 kg/m^3) to promote "autogenous healing" of cracks and construction joints.

(3) Small dosage of silica fume benefits strength and workability; large dosage impairs constructibility.

(4) High slump retention requires careful consideration of fine-aggregate grading and chemical admixtures.

(5) Full-scale site trial is essential before concrete mixtures and placement scheme are chosen and approved.

c. Employ modern automatic batching plants.

(1) Efficient mixing.

(2) Fail-safe computer control to eliminate errors.

(3) Printout of each batch.

(4) Watt-meter checks on workability.

d. Develop sound work procedures beforehand.

(1) Think concreting before laying out steel bars.

(2) Ensure backup plant and materials.

(3) Do trials or mockup if in doubt.

(4) Get commitment from the crew by participation in planning.

(5) Apply high-frequency poker vibrators in the correct manner.

e. Compact the concrete thoroughly.

(1) Apply high-frequency poker vibrators in the correct manner.

(2) Revibrate top layer for increased strength and elimination of voids under embedded items.

(3) Make sure the concrete cover is fully compacted (apply small-diameter poker vibrator if necessary).

f. Ensure adequate cover to rebars.

(1) Minimum 50-mm cover to main reinforcement.

(2) Quality of the cover is as important as thickness.

(3) Posttensioning anchorage must be recessed and have rebar tiebacks covered with high-quality concrete patch.

g. Pay attention to construction joints.

(1) Select construction joints with careful consideration of the exposure condition of the environment, the potential stress level, and stress variations at the joints. Joints should be located in zones where the exposure to saltwater is minimal and potential tensile stresses are minimal. Stress reversals in the joints should be voided.

(2) Proper construction procedure (removal of laitance, application of rich cement mix over joints, thorough vibration, etc.).

(3) Slipforming can give monolithic walls without construction joint.

h. Make allowance for temperature.

(1) Avoid concrete curing temperature above 70 °C.

(2) Avoid excessive temperature difference across section.

(3) Heat of hydration in mass concrete generally causes 12 °C temperature rise per 100 kg cement. High content of fly ash and/or slag should be used to avoid high portland cement content.

(4) Initial concrete temperature at placement can be efficiently reduced by ice flakes as part replacement of mixing water (8 kg/m^3 ice give 1 °C temperature reduction).

(5) Never grout posttensioning ducts without making sure that the ducts are free of ice.

i. Keep design simple and effective.

(1) Generous sections facilitate concrete placement.

(2) Rounded and smooth surfaces discourage local concentration of deleterious materials and corner cracking.

(3) Minimize large horizontal surface and recess in the splash zone.

(4) Avoid sudden changes in geometry.

(5) Larger steel bars take less space.

(6) T-headed bars reduce stirrup congestion.

(7) Cathodic protection of exposed steel must allow for current drain to the embedded reinforcement.

j. Use trained and skilled operators. Only the operators performing the work can effectively and continuously affect the quality of the work.

Of critical importance is the construction requirement and detailing for prestressing strands. The installation, stressing, and grouting of the posttensioning tendons in offshore structures are usually more complex than for other types of

structures, because of varying cross sections, curvatures, and direction of prestress. Many of the tendons have multiple curves in more than one plane.

Tendons and posttensioning appurtenances for offshore structures are usually installed near or over salt water. More than usual care is required to protect tendons from saltwater contamination and mechanical damage during storage, transport, and installation. Specifications normally require that each coil or bundle of prestressing cables be sealed in a polyethylene bag or wrapping during storage. All wires or strands should be delivered with a protective coating on the steel. This coating must be removed prior to the installation of the tendons and grouting of the ducts. However, if water-soluble oil is used, it need not be flushed off prior to grouting if field testing shows satisfactory results. After installing and stressing of the tendons, but before grouting and prior to concreting the anchorage pocket, tendons should be protected from saltwater spray by a plastic cover.

Since field operations are mostly carried out afloat, the majority of the ducts are located near, or below, the external waterline. It is important to keep the duct clear of salt water. Should any entry of salt water occur due to splash, the duct must be washed out with fresh water.

It is essential to require and enforce that all prestressing ducts and sheathing be made watertight. The ducts should not be ruptured or cracked by accidental contact of the vibrators. This consideration leads to selection of heavy-gauge rigid metal or plastic ducts in offshore structures. The wall thickness of the metal ducts should not be less than 2.0 mm in the splash zone of the structure. Elsewhere, the minimum thickness of metal ducts may be 0.8 mm (black) or 0.6 mm (galvanized). All construction joint splices in rigid ducts are commonly bell and spigot. The spigot should lead in the direction in which tendons will be inserted. Sleeves should be used for joining semirigid and flexible ducts. All splices should be taped with waterproof tape.

Flexible ducts are used only in special areas where the rigid or semirigid duct is impracticable, such as at sharp bends. A mandrel should be inserted into the ducts to prevent them from dislocating during concreting.

Past experience show that ducts in offshore structures have experienced more than usual blockage. The causes of blockage are usually either entry of foreign objects (pieces of gravel or short bars) or leakage of cement paste at splices. The specifications commonly require that the open end of the ducts be capped with red plastic cover and heat-shrinkage tape used at splices.

The grouting procedure and arrangement of entry and exit ports should be designed to ensure complete filling of ducts, including forcing grout through wedges at anchorage. To ensure complete filling of ducts having significant vertical rise, full-scale testing is usually conducted prior to actually commencing the grouting.

The anchorage for prestressing strands has been a major concern for corrosion protection in the marine environments. Standard detailing against corrosion attack is normally followed in offshore structures. The anchorage is generally recessed and

protected by reinforced concrete. Exposed ends of strands protruding from the anchorage should have their ends sealed after grouting of the ducts. The surface of the anchorage pocket and the anchorage itself should be coated with a suitable bonding agent and the pocket filled with concrete. The concrete fill should be tied to the structure with reinforcement. The joints between the concrete fill and the base concrete should be painted with polymer coating. In the splash zone and other wetting-drying environments, a vapor-permeable material, such as latex, should be used as a bonding agent and coating.

Durability Performance of Offshore Concrete Structures

Since the installation of the first major offshore gravity-based structures (GBS)—Ekofisk in 1973—about thirty major concrete GBS platforms have been installed over the world. A majority of these platforms was installed in the North Sea. The average age of these platforms is about 20 years. Also, a large number of relatively small offshore concrete platforms and floating oil/gas stations have been built for production of oil and gas. The majority of these structures were prestressed to ensure their strength and durability requirements. Over the years, these structures have been under periodic inspections and, in some cases, systematic monitoring. The purposes of the inspections and monitoring are somewhat different for the various platforms, including:

a. Confirmation of design assumptions and analysis results.

b. Confirmation of structural safety after many years of wear and tear.

c. Investigations related to accidental damage and subsequent repair.

d. Requirements of regulatory authorities.

As a result, the extent and the procedures of these inspections vary considerably. So far, the only main source of the relevant information on the platform performance is the inspection reports of the oil companies. Unfortunately, these proprietary data cannot be cited in this report.

In 1991, Ben C. Gerwick, Inc., investigated hundreds of offshore concrete platforms and floating facilities for Norwegian Contractors (Ben C. Gerwick, Inc. 1991a). In 1995, Federation International Precontraite (FIP) performed an overall evaluation of the concrete GBS platforms in the North Sea (FIP 1996). These published documents have, to a large degree, been substantiated by many proprietary inspections and monitoring. The main findings of these two studies are cited in this section.

Between late 1950s and the 1990s, over 1,000 concrete platforms have been put into operation in the Gulf of Mexico. These structures are mostly bottom-founded concrete barge platforms that were floated into position and then setdown on the

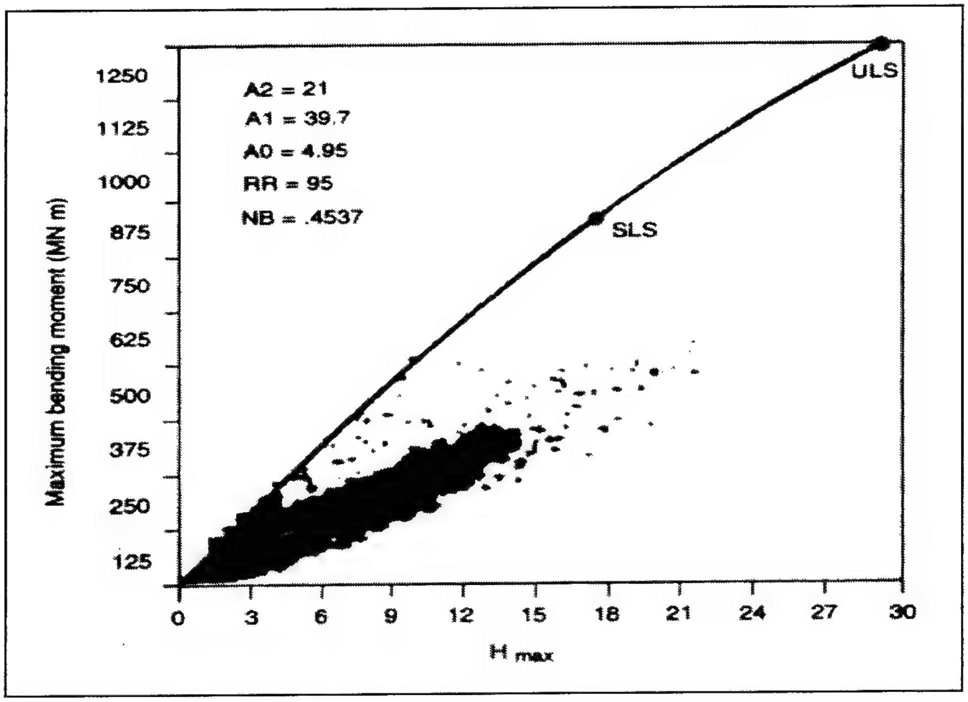
bottom to serve a variety of functions, such as gas compressor stations and storage facilities. The majority of these platforms are operated in brackish water marshlands along the coast. Therefore, the single largest durability concern for these concrete structures is clearly that of corrosion of the reinforcing and prestressing steel. The hot, humid, salt-laden environment of the Gulf of Mexico is conducive to corrosion if the concrete cover does not maintain its integrity against the migration of salts, water, and oxygen.

Over the last 3 decades, these structures have received little or no maintenance. Nevertheless, nearly all of these oil/gas concrete platforms are still in operation. Some of these structures have been upgraded and redeployed over the period. A noteworthy case is the ARCO's concrete LPG (liquefied petroleum gas) terminal, Sakti Ardjuna off, the north coast of West Java in Indonesia. The Sakti Ardjuna is one of the largest offshore floating concrete terminals built in the United States. The structure was prefabricated in Tacoma, Washington in 1976, following American Building Standards and American Concrete Institute (ACI) guidelines. It is 140 m long, 41.2 m wide, and 17 m deep. About 25,000 tons (222 MN) of concrete was cast in the structure, along with approximately 1,600 km of 0.5-in. (13-mm)-diam seven-wire strands. Concrete has a of 0.4. The average 28-day compressive strength was closer to (67 MPa). The 1986 and 1996 surveys showed that there was no significant deterioration to the structure over the years, and there was no need to dry-dock the structure for maintenance.

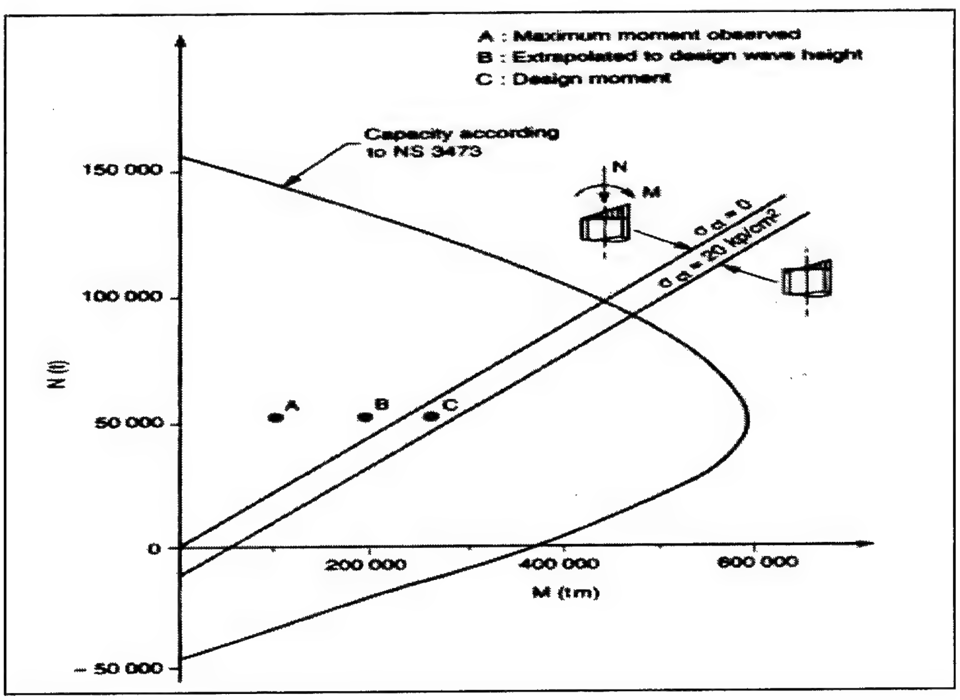
The major concrete GBS platforms in the North Sea exhibit remarkably sound and reliable behavior in a very hostile environment. In general, the performances of these structures has not been severely impaired in decades of exposure to huge waves, strong currents, violent storms, and aggressive attack of seawater. No significant signs of material deterioration, corrosion of reinforcement, or other material deficiencies have been observed in the submerged zone. The structural integrity of these platforms has never been jeopardized. Typical damages have been impacts from ships and dropped objects, resulting in local cracking of the concrete. A majority of these platforms have been fitted with strain measurement systems to determine whether the long-term structural behavior complies with the design assumptions. Figure 10 shows a typical relationship between the measured shaft moment and the design capacity. The measurements remain small relative to the design capacity. As usual, the design is governed by the serviceability requirements for crack control.

During the main construction phases, the platforms normally float at an inshore construction site. The lower parts of the structure are exposed to the hydrostatic water pressure. Minor leakage had been often observed in these phases. Before the platforms are towed out and installed, these leaks are routinely eliminated by epoxy injection.

During the operation phase, minor water or oil leakage has also been observed in some of the shafts. These leaks are almost exclusively found at construction joints and pipe penetration. Concrete cast without such disturbance has never shown any sign of leakage or free moisture. A number of major leakages of water or oil



a. Maximum bending moment at base of shaft versus wave heights



b. Measurements versus design values and capacity

Figure 10. Relation between measured shaft moment and design capacity

have occurred. In all these incidents, the cause of the problem was improper construction of cold joints.

In summary, offshore prestressed concrete structures, either fixed or floating, have achieved excellent durability performance in very severe exposure environments, provided that modern design criteria are imposed and construction is properly executed.

4 Design Criteria for Prestressed Concrete Offshore Structures

Design criteria for offshore and marine structures vary greatly depending on the type and functionality of the structures considered, and the environment to which the structures are exposed. Design criteria for pile-supported marine terminals are substantially different from those for offshore gravity-based structures as oil drilling platforms. Piers and wharves are generally frame structures in sheltered water, while offshore platforms are primarily complex multicell shell structures. Piers and wharves are primarily subjected to cargo and crane loads on deck, while offshore platforms are subjected to lateral cyclic loading, such as 40-m waves and impact from multimillion tons of iceberg. Construction of the waterfront structures adopts land-based construction techniques, while construction of offshore platforms is completely marine operations.

A comprehensive review covering design criteria for all types of offshore structures is beyond the scope of this study. Instead, this report is intended to illustrate the essential features of offshore structure design by focusing on one group of marine structures—the offshore GBS platforms.

In general, the design and construction of major offshore GBS platforms in the North Sea and the Arctic Ocean represent some of the most creative engineering and sophisticated technologies in marine civil works projects today. In many ways, construction of these platforms has similar characteristics to the in-the-wet construction of navigation structures. In both cases, a major portion of the structure is prefabricated offsite and then transported over water to the site for installation.

The design of offshore GBS platforms includes a sequential determination of loads, load effects on the structure, resistances of the structure, and a final safety check of various limit states considered. Therefore, the design criteria must define the loads and load combinations: Ultimate Limit State (ULS), Serviceability Limit State (SLS), and Fatigue Limit State (FLS). Furthermore, the detailing and construction requirements of prestressed concrete are essential parts of the design requirements. These criteria and requirements generally govern the design of offsite prefabricated structures and will be discussed in the following sections.

In addition to the ULS, SLS, and FLS requirements, GBS platforms are usually checked against Progressive Collapse Limit State (PLS). The PLS corresponds to the condition that failure of one member due to accidental or abnormal overloads leads to progressive failure of adjoining members, and eventually to capsizing or collapsing of the structure. In general, PLS design is specific to each individual structure. Generalization of the PLS design criteria for GBS platforms to other types of structures may be of limited use. Therefore, PLS is not discussed in this report except for the general principles described below.

The design against progressive failure is usually achieved by a combination of the structural strength and ductility. The ductility requirement is to allow for large inelastic deformation of structural members so as to absorb the energy from very high loading in an accidental event, such as ship impact or earthquake. Ductility should be defined both at the structural element level and the structural system level. At the element level, individual members should be designed in such a way that the reinforcement governs the load-carrying capacity. Brittle failure, such as compression failure or shear failure of concrete, should be avoided. At the level of the structural system, structures should be designed to achieve large energy absorption and dissipation capacity. The potential causes of brittle failure should be eliminated. For example, the design philosophy of weak beam and strong column is generally recommended to limit potential damage in local zones of secondary structural members so as to avoid general collapse. Abrupt changes in structural stiffness should be avoided. In critical zones, multiple load paths should be provided to ensure structural redundancy.

Load Criteria

Offshore GBS platforms are commonly designed with the limit states design method, or the "partial coefficient method" as it is known in Europe. In this design method, the strength of the structures is investigated by comparing factored load effects to factored resistances (ULS). The serviceability of the structures is investigated by comparing the effects of service loads to acceptable limiting values (SLS). In addition, desirable durability and ductility are obtained by complying with detailing requirements and engineering specifications.

The factored load effects are obtained by multiplying characteristic loads with their corresponding load factors. In the design of offshore GBS platforms, characteristic loads are commonly classified in five general categories as follows:

a. Permanent load (P), including dead load, permanent ballast and equipment, and hydrostatic load of permanent nature.

b. Live load (L), including deck load from appurtenances and stored materials, helicopter and crane loads, and temporary load associated with construction and installation (such as towing, anchoring, and lifting operations).

c. Deformation load (D), including prestressing forces, thermal load, and load effects due to shrinkage, creep, foundation settling, and uneven seabed.

d. Environmental load (E), including wave and ice load, dynamic impact from iceberg and boats, and earthquake load.

e. Accidental load (A), such as accidental loads from dropped objects, ship impact or helicopter collision, explosion, or fire.

The basis for selection of characteristic loads for both temporary conditions and operation conditions is specified in offshore design codes (e.g., Det Norske Veritas (DNV) 1997, Canadian Standards Association (CSA) S474-94). As an example, DNV establishes the criteria for selecting characteristic loads as shown in Table 2. In the table, "expected value" of a characteristic load can generally be interpreted as the mean value of the load, and "specified value" can be interpreted as maximum or minimum probable value of the load during the period considered, taking into account operational requirements and the required safety.

Table 2
Basis for Selection of Characteristic Loads in the DNV Code (after DNV 1997)

Load Category	Ultimate Limit State	Fatigue Limit State	Progressive Collapse Limit State		Serviceability Limit State
			Intact Structure	Damage Structure	
Temporary Design Conditions					
Permanent (P)	Expected value	Expected value	Expected value	Expected value	Expected value
Live (L)	Expected value	Expected value	Expected value	Expected value	Expected value
Deformation (D)	Expected value	Expected value	Expected value	Expected value	Expected value
Environment (E)	Expected value	Expected load history	Expected value	Expected value	Expected value
Accidental (A)	Not Applicable	Not Applicable	Not Applicable	Not Applicable	Not Applicable
Operation Design Conditions					
Permanent (P)	Expected value	Expected value	Expected value	Expected value	Expected value
Live (L)	Specified value	Specified value	Specified value	Specified value	Specified value
Deformation (D)	Specified value or as given for environmental load category	Specified value or as given for environmental load category	Specified value or as given for environmental load category[2]	Specified value or as given for environmental load category	Specified value or as given for environmental load category
Environment (E)	Annual probability[1] being exceeded = 10^{-2}	Expected load history	Annual probability[1, 2] being exceeded = 10^{-4}	Annual probability[1] being exceeded = 10^{-1}	Specified value
Accidental (A)	Not applicable	Not applicable	Annual probability[2] being exceeded = 10^{-4}	Not applicable	Not applicable

[1] The probability of exceedance applies as stated in Sec. 3 A400.

[2] The joint probability of exceedance of combined load categories is not required to be less than 10^{-4} (see A403).

It is recognized that environmental loads are mostly random in nature and their characteristic values should be the most probable largest values in a time period equal to the design period for the load. In general, the recurrence interval for all environmental loads under service conditions is 100 years or more. However, for

temporary exposures such as construction, towing, and installation of the structures, the recurrence interval of the extreme environmental event can be reduced to account for the actual exposure period and season in which the operation takes place. The relevant environmental loads during construction and installation are usually determined on the basis of available statistical data in the specific region, towing route, and the season of the year in which the construction takes place.

For the SLS and the FLS, the environmental loads should include only the loads associated with wind, wave, current, and ice. All the load factors should be 1.0, except that the load factors of environmental loads are taken as 0.6, because these characteristic loads are taken as "specified value" in Table 1.

The effect of accidental loads should be limited to local damages, and the remaining structure can sustain the loading conditions likely to occur prior to full repair of the local damage. When the ULS is checked against accidental loads, the load factors for load types P (permanent), L (live), E (environmental), and A (accidental) are taken as 1.0. The load factor for deformation load due to prestressing and temperature effects is also 1.0. Other deformation loads can be neglected, except that the evaluation of shear capacity should account for all the axial deformation loads.

For ULS design, load factors are specified for each specific load combination. Since the offshore GBS platforms are prefabricated offsite and then transported and installed in deep water, they are subjected to many loads that vary in magnitude, location, direction, and duration during different load stages. The critical load combinations should be determined for five general load stages, as listed below:

a. Construction stage, including mooring in protected waters, concrete construction, and deck mating.

b. Transportation stage, including launching from shore to sea and towing completed structures to the final location.

c. Installation stage, including positioning and ballasting of the structure.

d. Operation stage, including the entire service life of the structure.

e. Removal stage, including reflotation and disposal of the structure.

Thorough engineering analysis must be conducted for all the critical load cases and combinations during the stages of construction, transportation, installation, and operation. In each stage, the design loads should be combined in the most unfavorable, and physically possible, manner. Each individual member must be checked for both ULS and SLS for all the critical load combinations. The overall structural system should be checked for ductility and redundancy. In addition, the structural elements subjected to significant cyclic loads should be checked for FLS in the operation stage.

To demonstrate the significance of various load stages in the design, a general sequence of construction and installation of a typical offshore platform is presented below with illustrations in Figures 11-15. To give the overall view of the construction process, the numbers of construction stages are purposely abbreviated. In fact, there are numerous important substages in each main stage. More detailed discussions can be found in special technical publications (Gerwick 1986, 1993).

The process to construct a typical offshore GBS platform starts with construction of its base raft in a graving dock. The concrete base slabs are usually 1 to 2 m thick, posttensioned with long tendons to about 3 to 4 MPa prestress. Although the offshore GBS base rafts are extremely massive, they are flexible structures that will float over water. The prestress in the base raft is primarily responsible for resisting the cracking moment that will be imposed onto the slab during construction and installation of the platform. Throughout the construction stage, the actual initial prestressing force should be used in the design. If a lower prestressing force will create an unfavorable load combination, a factor of 0.88 is commonly applied to the initial prestressing force.

Once the base section is completed, the graving dock is flooded and the base raft is towed to an inshore deepwater site that is sheltered from major storm waves. The rest of the platform will be constructed in the sheltered water while the incomplete structure is moored and in a state of flotation. Floating out of the graving dock has to be carefully controlled because the typical raft is inherently unstable and weak in bending. In the float-out stage, the cantilever extension of the base raft toward the center tends to give a significant hogging moment. The unequal moment is often aggravated by the weight of skirts and cranes that are usually concentrated around the outside edges of the base. Effective measures must be taken to minimize the hogging moment. If the GBS base raft is not symmetrical, it is necessary to check critical bending moments along a number of possible axes. Potential accidental conditions must be carefully considered in the process, such as the loss of compressed air from under the base skirts, a broken ballast pipe, or rupture and flooding of one compartment. For these potential accidents, structural integrity, stability, and buoyancy of the raft must be maintained with minor distress in local zones.

Once the base raft is moored at the sheltered deep water, the rest of the platform is constructed over the base afloat. The moorings are designed to hold the substructure against strong winds and waves in any probable storm during the construction period. The cells and shafts are commonly constructed by slipforming, as shown in Figures 12 and 13. A typical base raft has about 1,200 linear metres of prestressed concrete walls with an average thickness of 0.8 m. Thus, a continuous placement of about 1,000 m^3 of concrete and 300 tons of reinforcing steel per day, along with prestressing ducts and embedments, will be required to raise the walls at a rate of 1 m per day. This will require up to 600 workers per shift, three shifts per day, for up to 6 months of continuous operation.

As the walls rise up, it is critical to examine each of the incremental steps of construction. The step-by-step addition of concrete changes weight, trim, and draft of the structure, which in turn affect the ballasting, stability, hydrostatic loads, and

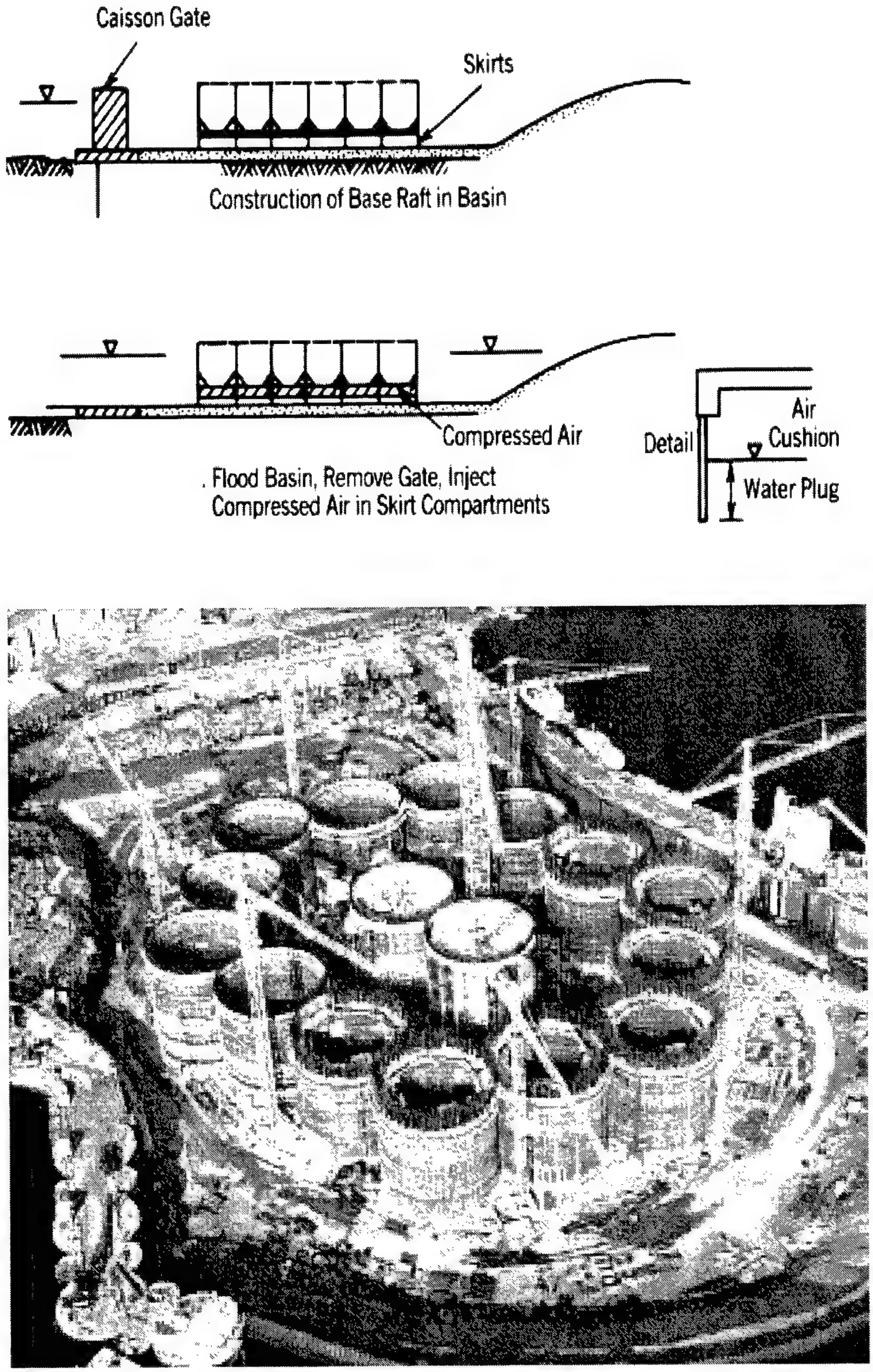


Figure 11. Construction of concrete base section in a drydock

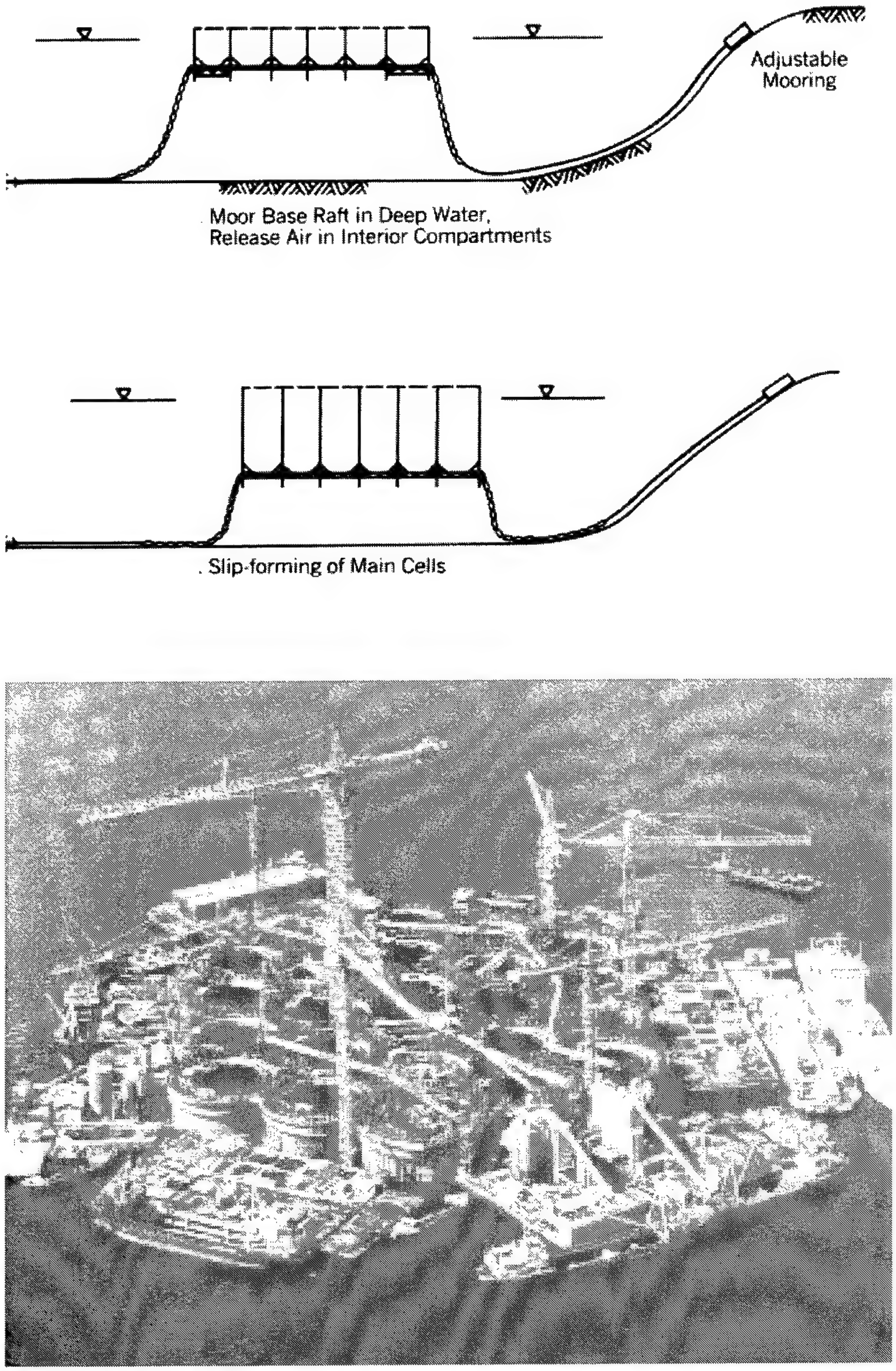
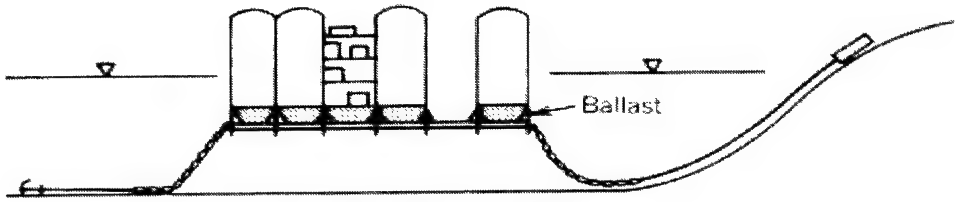
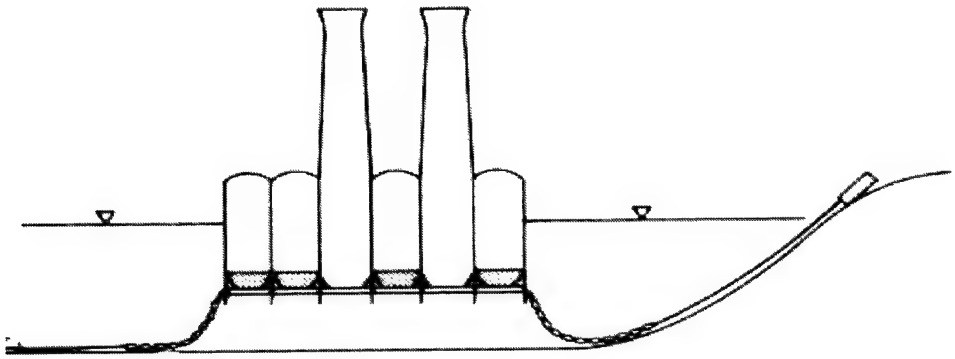


Figure 12. Construction of cell walls in sheltered water



Construct Upper Domes, Place Solid Ballast, Install Equipment in Utility and Riser Shafts



Slip-form shafts

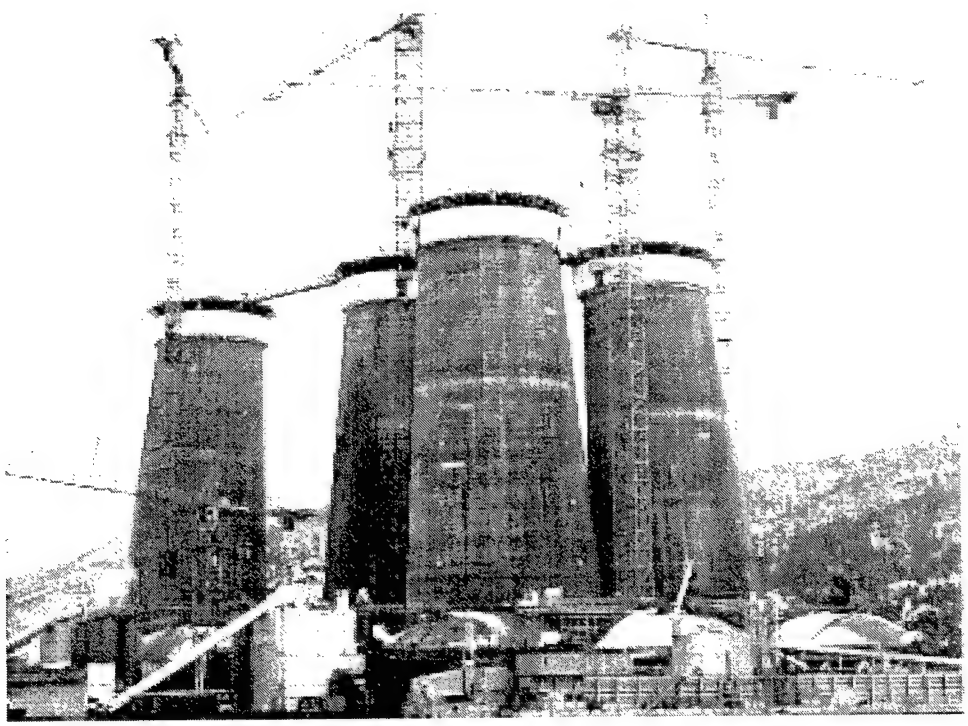
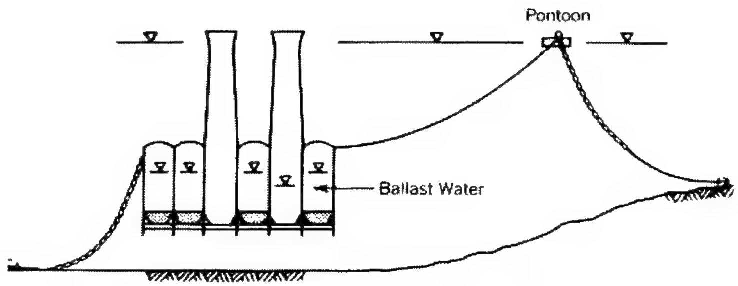
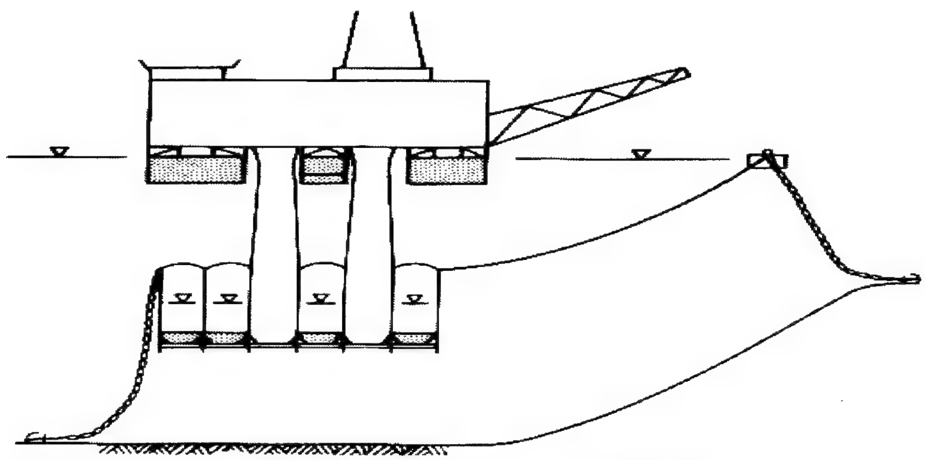


Figure 13. Slipforming shafts in sheltered water



Moor Substructure at Deep-Water Mating Site, Ballast Down to Minimum Freeboard



. Maneuver Deck over Substructure and Transfer Deck to Substructure

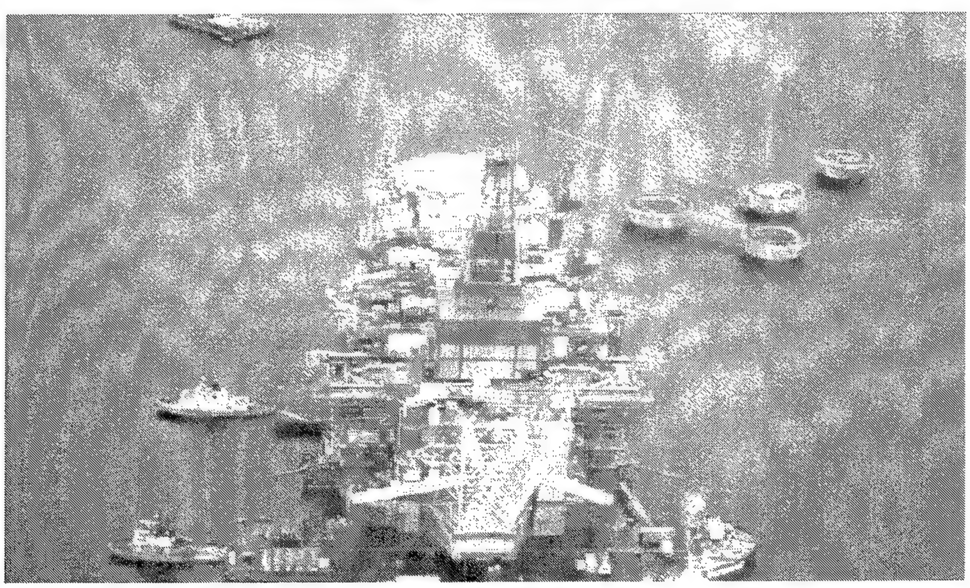


Figure 14. Mating of the platform with the deck (concrete platform is ballasted to minimum freeboard)

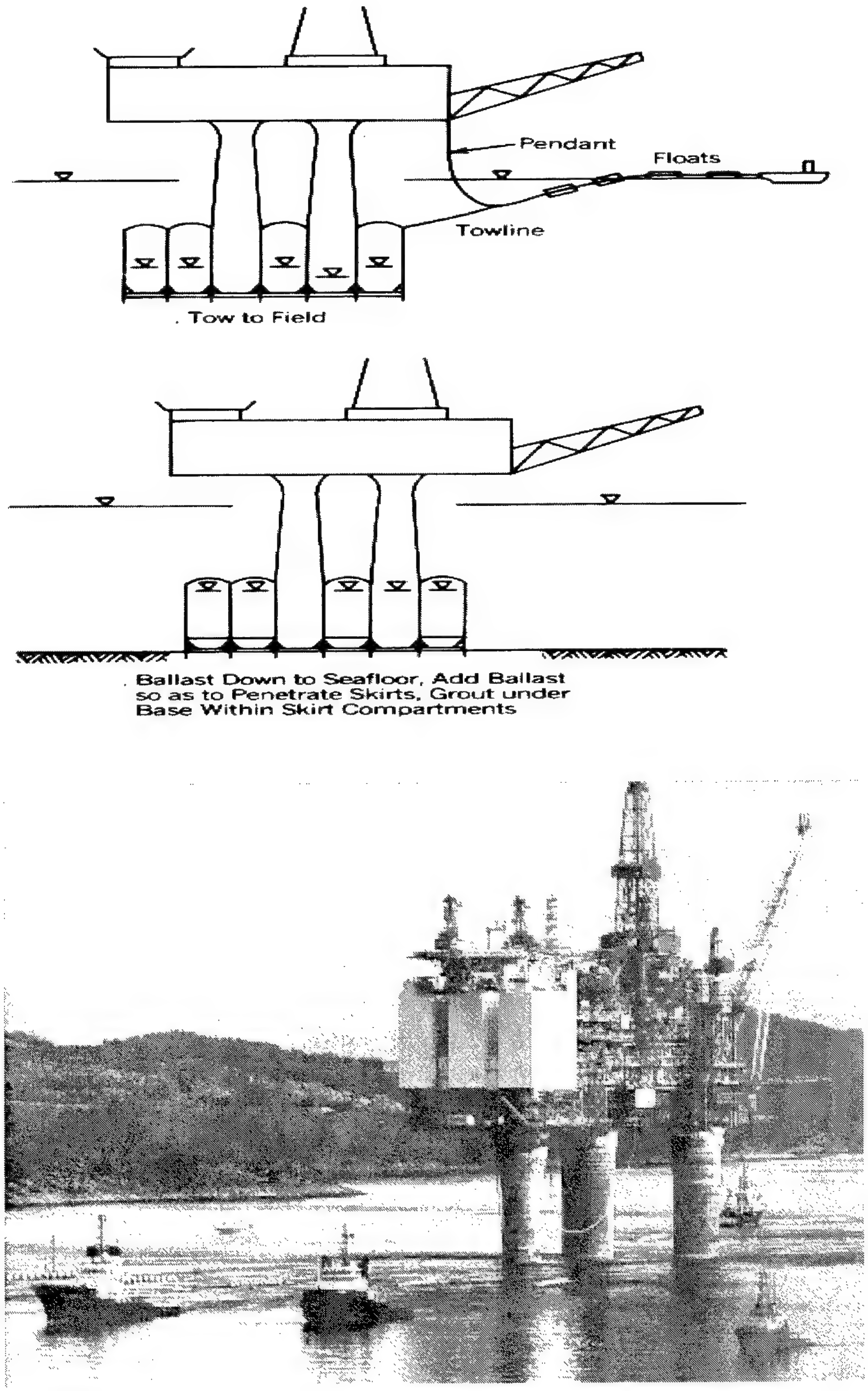


Figure 15. Towing out the platform to the field for installation

structural response to various load combinations. Selective ballasting of cells is conducted to keep freeboard level just a few metres above the water. The differential hydrostatic heads of ballast water in two adjacent cells must be carefully controlled and monitored. In checking the strength of cell walls against the differential water heads, an appropriate load factor should be applied to the difference between the two hydrostatic pressure heads. Solid ballast, such as concrete, is sometimes placed in the cells to substantially lower the center of gravity. Mass concrete will generate high heat of hydration, leading to thermal expansion, which may adversely load the adjacent walls and crack the base slab. Such thermal loads must be accounted for in critical load combinations.

Circumferential prestressing is usually required in ring beams at the top of the shafts and at the intersection between the cell domes and walls. The prestressed concrete ring beams at the top of the shafts must be constructed to a tolerance of a few millimetres for mating with the top deck. The deck structure, weighing up to 70,000 tons, is usually very heavy steel trusses and frames supporting various modules of equipment. The deck is fabricated in a shipyard and towed to the mating site on barges. The concrete substructure is ballasted so that only 3 to 5 m of shaft extends above the water, as shown in Figure 14. The deck on catamaran barges is slowly winched in around the shafts, with clearance of only 30 to 60 cm. Once the horizontal positioning is verified, the concrete substructure is deballasted to support and lift the deck up off the barges. The entire operation requires very careful planning and accurate field control to prevent overstressing the mating surface. The deck is then secured to the shaft by prestressing. Prestressing is especially effective in eliminating cyclic fatigue and in providing safety against uplift under accidental conditions.

Following mating of the deck and hookup of all equipment, the completed platform is towed through restricted water and open sea to the field, as shown in Figure 15. During the tow, stability is a controlling parameter. The draft, ballast, and payload of equipment are selected to give 1 to 2 m of positive metacentric height. Dynamic response of the platform's motions must be carefully determined under the design storm, which usually has the return period of 10 to 25 years. With its relatively low metacentric height, the structures typically roll and pitch in a relatively long period, developing maximum acceleration of about 0.25 g.

Arriving at the installation site, the platform is held in position by the tugs and mooring lines. Then, it is ballasted by pumping seawater into various cells and shafts. As the platform descends near the seafloor, the speed of descent is slowed to allow the water under the structure to escape. The short-range, high-frequency echo sounders on the platform constantly give the distance between the base slab and seafloor until the touchdown of the platform. The major platforms installed in the North Sea have typically been installed to an accuracy of a few metres.

In summary, the construction and installation of an offshore GBS platform involves many complex load stages, ranging from construction of the base draft in a dry dock to ballasting of the completed structure to the seafloor. The critical load combinations of the structure vary greatly in magnitude, location, direction, and duration from one load stage to another. Each main load stage also includes

numerous substages where construction and environmental loads on the structure can have significant variation. The design criteria must address all the critical loads in each load stage and substages. In the past, most errors have been the result of either overlooking an intermediate sub-stage or combining two or more substages to save computational efforts. Detailed sketches of each load stage and substages, along with evaluation of the relevant hydrostatic, environmental, and structural loads, must be prepared to enable visualization for structural analysis, design and construction planning.

Crack Control Criteria

Serviceability requirements usually cover two areas of structural behavior: (1) crack width and deflection and (2) durability performance of the structures. This section mainly addresses the design criteria on crack width control. General durability requirements were discussed in Chapter 3.

Objective and significance of crack control

The limiting of crack widths achieves the following objectives:

a. Acceptable level of corrosion resistance.

b. Durability by limiting ingress of aggressive agents.

c. Intended structural behavior (e.g., transfer of membrane shear) and fatigue resistance.

d. Watertightness.

In relation to the durability design of offshore concrete structures against corrosion attack and other deleterious actions by seawater, crack control has been subjected to extensive research as well as intensive debates in the past. Extensive studies include the U.S. Army Engineer Waterways Experiment Station's long-term exposure tests (Roshore 1971), the U.S. Navy's concrete durability study Haynes 1980), the British Department of Energy's "Concrete in the Ocean" program (Wilkins and Lawrence 1980), and Norwegian offshore and coastal field surveys of marine structures (Gjorv 1968, 1996).

In general, cracks will lead to initiation of corrosion by allowing deleterious materials (salts, water, oxygen, etc.) to easily penetrate into the interior of the concrete. The corrosion in turn promotes further cracking of the concrete through the mechanism of metal expansion. The crack-corrosion-crack phenomenon develops into a self-feeding cycle that has been primarily responsible for the most severe corrosion damage in offshore concrete structures. Prestressing tendons are especially sensitive to corrosion attack because prestressing steel is pretreated and/or cold worked. Prestressing steel may fail due to corrosion in a very brittle manner without much warning. However, prestressing tendons have had very little

corrosion problem in offshore structures, because they are encapsulated in grouted ducts within concrete members.

The susceptibility of cracked pretensioned concrete to corrosion attack was clearly demonstrated by a long-term exposure test (Roshore 1971). In 1959, three pretensioned concrete beams (114 by 229 by 2,057 mm) in size, were installed at a concrete durability exposure station at Salt Run, off Anastasia Island near St. Augustine, FL. The concrete had a design strength of 41.4 MPa with air-entrainment of 4.5 percent. Each beam was prestressed with nine 0.25-in. diameter (6-mm)-diam strands to 9.7 MPa of prestress and 0.011 MN-m of moment. The specimens were placed at the half-tide elevation on a timber wharf exposed to twice-daily tidal inundation. Two of the beams were loaded to crack. The third beam was not cracked as a control specimen. After 9.5 years of exposure, the beams were recovered and examined. The examination indicated that the two cracked specimens were corroded from 56 to 100 percent of the length of the beam, and the majority of the corrosion was associated with water ingress to the strands through the cracks.

Although the crack width does influence the time to initiation of the corrosion, extensive research and practical experience indicate that the crack width is of secondary importance to the rate of corrosion in comparison with concrete quality and concrete cover (Wilkins 1980, Mehta and Gerwick 1982, Beeby 1983, Brook and Stillwell 1983, Gautefall 1983). It is generally not the crack width but the area covered by the cracks that has significant effect on serious corrosion damage.

In general, extensive investigations to date have not led to quantitative conclusions on the correlation between the crack width and corrosion damage in prestressed concrete. However, general qualitative conclusions can be drawn from the relevant experimental database and field observations as follows:

a. Concrete cracks along reinforcing steel bars impose serious risk of steel corrosion.

b. Concrete cracks transverse to the main reinforcing steel bars tend to accelerate the initiation of steel corrosion. Once corrosion begins, however, the rate of corrosion may not be directly related to the transverse crack width.

c. The submerged zone of the offshore structure usually does not have significant risk of corrosion due to the lack of oxygen in water.

d. The splash zone in marine environments is subjected to very corrosive exposure. Adequate preventive measures should be taken against corrosion attack, including an increase in concrete cover, reduction in concrete permeability, and control of crack width.

e. The practice of using smaller steel bars and closer spacing to controlling crack width should be applied with caution. While the initiation of corrosion at a particular crack may be reduced slightly by reducing its crack

width, the overall risk of significant corrosion actually increases in proportion to the increase in the number of bars.

f. If cracks have to be controlled to avoid corrosion damage, the control will best be achieved by a reduction of the steel stress rather than the use of smaller bars or spacing.

Transverse cracking of concrete often plays an important role in fatigue design of offshore concrete structures. Reinforced concrete structures are not effective in resisting diagonal cracks under cyclic in-plane shear loading due to the orthogonal reinforcement layout. Once the cracks open, the potential damage due to water pumping in cracks can lead to accelerated deterioration of submerged concrete. The current design philosophy is to minimize the possibility of such damages by limiting crack occurrence. For critical members subjected to repeated loading, fatigue design criteria commonly require that no membrane tension be allowed and edge stresses due to bending be limited below the modulus of rupture. These requirements essentially impose a limitation of no through-cracking by means of prestressing. Prestressing in one direction can change the inclined direction of the potential membrane shear crack to the orthogonal direction so that the conventional reinforcement is more effective. Prestressing in both directions further reduces crack potentials.

Leakage through concrete cracks is an important design consideration if there is a differential hydrostatic pressure on two sides of a concrete wall. Water tightness is of main interest in buoyancy chambers, oil storage tanks, and hollow leg structures where assumed pressure differences must be kept. A few incidents of minor leakage have historically been acceptable. Crack widths are generally checked at critical locations against static loads plus 40 percent dynamic loads.

Numerous laboratory experiments and field measurements (FIP 1985, Aker Norwegian Contractors 1996) indicate that a partially cracked concrete section with a compression zone of no less than 30 mm is essentially free of leakage. Consequently, offshore design criteria normally require that any section of a watertight component in offshore GBS be designed to have a compression zone no less than 30 mm and 0.25 h in thickness (h is the overall thickness of the components). In addition, the watertight components are generally required to maintain a minimum compressive stress of 0.5 MPa across the section under service loading conditions. The compression should account for thermal stresses. For temporary loading conditions during construction and installation, the stresses may be increased to 2 MPa in tension.

It has been suggested that water flow through concrete cracks could cause deterioration by leaching and reduction in corrosion resistance. In practice, however, observed leakage has mostly been caused by construction defects, mainly in construction joints. Cracking caused by simplified or relaxed design criteria for crack control has not been seen to cause leakage. Concrete structures in seawater can gradually seal minor cracks over time by a so-called "autogenous healing" process.

When applying a general code, such as ACI 318, to the design of offshore concrete structures, there will generally be some serious contradiction between crack control provisions in the codes and the performance requirements of the structures. Therefore, substantial experience and engineering will be required in specifying the crack criteria for offshore structures due to their special requirements and environmental conditions. In principle, the crack control criteria for offshore concrete structures should be based on at least four important factors:

a. The performance criteria and environmental exposure of the structure.

b. Concrete cover to reinforcement.

c. Concrete quality.

d. Minimum reinforcement to limit crack width.

Minimum reinforcement for crack width control

The concept of minimum reinforcement for crack control is introduced in offshore codes to control the cracks resulting from all the stresses that are not covered by the ULS design and crack width calculation. The amount of minimum reinforcement should be designed to ensure that another crack is formed before the reinforcement at an earlier crack yields. Implementing minimum reinforcement allows the design to

a. Achieve a crack development with well-distributed fine cracks rather than widely spaced large cracks.

b. Ensure a ductile behavior by providing enough reinforcement to prevent an immediate and brittle failure when the concrete is cracked.

c. Avoid unreasonably large stiffness reduction in the transition from uncracked to cracked conditions so as to achieve ductility.

d. Ensure a controlled crack development when the structure is exposed to internal and external restraining forces.

e. Provide proper robustness in absorbing unforeseen exposure or local construction defects without severe cracking.

Nearly all the design codes and criteria for offshore concrete structures impose the minimum reinforcement requirements (A_s) in the same basic form:

$$A_s = k\frac{(f_{cr} + w)}{f_y}bh_e \qquad (1)$$

where

k = modification factor to account for variations in the concrete strength and the crack widths

f_{cr} = modulus of rupture

w = hydrostatic pressure of water in the crack surface

f_y = yield strength of reinforcement

b = width of the member

h_e = effective height of member

Concrete area (bh_e) represents the effective tension zone of concrete that can influence the crack width. The actual definition of the zone varies slightly for different design codes. The American Concrete Institute (ACI 357R-78) recommends that h_e be taken as $1.5c + 10d_b$ and $0.2h \leq bh_e \leq 0.5(h-x)$. Variable d_b is the average diameter of the steel bars. However, most offshore design codes impose a slightly less requirement: $h_e \leq 1.5c + 7d_b$.

Once the concrete cracks, water penetrates the crack. Water pressure in the cracks cannot generally be ignored and will contribute significantly to the minimum reinforcement requirement for concrete submerged in deep water. The hydrostatic pressure of the penetrating water will resist the formation of new cracks and enlarge the existing crack. The pressure is consequently added to the concrete tensile strength in the equation.

Design methods for controlling crack width

There exist two distinctive approaches toward crack control design in offshore concrete structures. The first approach is to control the width of individual cracks by imposing allowable limitations on the tensile stresses in reinforcing and prestressing steel. In this method, the tensile stresses in reinforcement under the service load condition are checked against a set of established allowable stress criteria. If the steel stresses are found within the stress criteria, the crack width is assumed to be acceptable.

The second approach is to calculate the actual crack width and compare the calculated crack width with a specified crack width criterion. Different crack widths will generally be specified for different structures and for different areas of the same structure, depending on the performance requirement, exposure conditions, and loading condition.

These two crack control methods are discussed in the following sections.

Allowable stress criteria for crack control

In the early offshore GBS design, crack control criteria were established on the basis of limitations of the steel stresses. The allowable stresses in reinforcing steel were typically limited to about 160 MPa in the submerged zone and about 100 MPa in the splash zone. Past experience has shown that, under most circumstances, the stress limitation design criteria not only provide effective crack control, but also simplify the design. The adequacy of these design criteria for the early offshore concrete structures has been demonstrated by the excellent service performance of these structures in severe environments over the last 20 years.

Based upon past experience with crack control in offshore concrete structures, ACI 357R recommends comprehensive stress limitation criteria to control the stress range in prestressing steel (Δ_{ps}) and the tensile stress in reinforcing steel (f_s), as shown in Table 3.

Table 3
Crack Control Criteria—Allowable Tensile Stresses for Reinforcing Steel and Allowable Tensile Stress Ranges for Prestressing Steel (after ACI 1984)

Stage	Loading	Δps	f_s
Construction: Where cracking during construction would be detrimental to the completed structure	All loads of the structure during construction	130	160
Construction: Where cracking during construction is not detrimental to the completed structure	All loads on the structure during construction	130	210
Construction	All loads on the structure during transportation and installation	130	160
At offshore site	Dead and live loads plus monthly recurring environmental loads	75	120
At offshore site	Dead and live loads plus extreme environmental loads		0.8 f_y

Since mid-1980s, the offshore industry has witnessed a trend of tightening the crack control design criteria. The allowable crack width criteria are almost exclusively used for the crack control check. The allowable crack width criteria are almost exclusively used in the serviceability check. This trend has been supported by sophisticated theories for crack width analysis and by sophisticated computer analysis. Analysis and design for SLS, particularly the crack width control requirement, is often performed by a "point-to-point" check. Sectional forces from every single element of a global finite element analysis have to be analyzed to document the SLS.

In the authors' opinion, the recent trend toward more complex and stringent crack control criteria has not always resulted in cost-effective design. The point-to-point check often lead to problems with unrealistic peak stress values, such as stress

concentration at corners. The computer analyses often pinpoint a nominal infringement of design criteria that has no significant consequence to the structural safety, serviceability, and durability. In many cases, the amount of reinforcement required for crack control is 15 to 30 percent more than that needed to satisfy the ULS requirements. Even more serious is the fact that the significantly increased density of reinforcement may aggravate corrosion by providing more cathodic voltage. It is questionable whether crack width design criteria with emphasis on difference of as little as 0.05 mm are a valid measure of corrosion susceptibility.

Allowable crack width criteria

Although calculation of crack width with a sophisticated mathematical procedure is not generally recommended for point-to-point checking of an entire structure, it is often necessary to selectively check the crack width in special areas, such as the splash zone, critical structural members susceptible to fatigue damage, or structural members that do not meet the stress limitations specified in Table 3.

It should be emphasized that the crack width criteria discussed in this section address only the cracks transverse to the reinforcing bars. Longitudinal cracks along the bars must be controlled by other preventive measures, discussed in Chapter 3 of this report.

In offshore structure design codes, the design criteria for crack width calculation typically consist of five steps:

a. Definition of limit state. The three are three basic limit states of cracking are as follows:

   (1) *The limit state of decompression* is the condition in which the compressive stresses due to prestressing are just eliminated at the fiber considered.

   (2) *The limit state of cracking formation* is the condition in which the tensile stress in the fiber considered reaches the characteristic tensile strength of the concrete.

   (3) *The limit state of cracking development* corresponds to the condition that cracks reach the allowable crack width defined in the specification.

b. Definition of environment. Three basic conditions of exposure are specified in terms of potential corrosion attack on reinforcing and prestressing steel: mild, moderate, and severe exposure.

c. Definition of loading conditions. The three basic loading conditions are permanent, frequent, and rare.

d. Definition of sensitivity of steel to corrosion. The steel used in structures is classified to be either very sensitive to corrosion or slightly sensitive to corrosion. The steel that is very sensitive to corrosion includes cold-worked

steel and pretreated steel, such as prestressing strands. Regular reinforcing bars are slightly sensitive to corrosion.

e. Definition of the allowable crack width. In each specific project, the allowable crack width should be specified on the basis of the environmental exposure, loading condition, and sensitivity of steel to corrosion as discussed above. For example, typical allowable crack width (W_k) limitations under frequent loading conditions in offshore GBS design specifications are as follows:

(1) Under sustained and operational loading conditions:

$W_k \leq 0.4\ mm$	in the submerged zone
$W_k \leq 0.2\ mm$	in the splash zone

(2) During construction, transportation, installation:

$W_k \leq 0.3\ mm$	for cracked-through section
$W_k \leq 0.5\ mm$	for partially cracked section

For prestressing tendons that are protected against corrosion, crack width limitation is the same as reinforcing steel. Corrosion protection of prestress tendons means multistrand tendons encapsulated in a plastic tube, or monostrand tendons protected with grease and extruded sheathing or equivalent. However, the allowable crack width for prestressing strands should be modified as

$$W_k' = \frac{1}{2} W_k \frac{\varepsilon_1}{\varepsilon_2} \tag{2}$$

where

$\varepsilon_1, \varepsilon_2$ = mean tensile strains (at cracked state) at concrete surface and at location of prestress steel, respectively

It should be emphasized that specification of the allowable crack width requirements should be based on other pertinent design parameters, such as the concrete cover, concrete quality, and diameter and spacing of steel bars. For offshore structures, the nominal minimum concrete cover for protection of reinforcing and prestressing steel is related to the zone of exposure. In this regard, the ACI 357R (1984) report and the European code (Comite Euro-International Du Beton (CEB) 1990) have similar requirements. In the submerged zone, the minimum cover is 50 mm over main reinforcing bars and 75 mm over prestressing tendons. In the splash zone and atmospheric zone subjected to sea water spray, the recommended minimum cover thickness is 65 mm for reinforcing bars and 90 mm for prestressing steel.

When the concrete cover used in the structures is greater than the minimum cover requirement, the allowable crack width may be scaled up proportionally by a factor of C/C_{min}. In other words, the characteristic crack width should be estimated as follows:

$$W_k = W_{0k} * \frac{C_{min}}{C} \geq 0.7W_{0k} \tag{3}$$

where

W_{0k} = calculated crack width at concrete surface

C_{min} = minimum required concrete cover

C = actual cover

This modification to the allowable crack width accounts for the facts that cracks tend to reduce in width toward the steel bars and that a large crack width with a large concrete cover is not more detrimental than a small crack width with a small cover.

Calculation of decompression and crack formation

To predict the limit state of decompression and crack formation, calculations can be carried out on the basis of elastic mechanics. In principle, checking stresses due to normal loading is very straightforward. For example, beams and one-way slabs can be checked for crack formation with the following equations:

$$\sigma = \frac{P_e}{A} \pm \frac{M_y}{I} \quad \text{for axial force and bending moment} \tag{4}$$

$$\tau = \frac{VS}{Ib} \quad \text{for shear} \tag{5}$$

where

σ = flexural stress

P_e = normal force (effective prestressing force)

A = cross-sectional area

M = bending moment

y = distance to extreme inertia

I = moment of inertia

τ = shear stress

V = total shear force

S = statistical moment of the cross-sectional area

b = width of member

Due design consideration, however, should be given to a number of load-independent deformations, such as those due to creep, shrinkage, and temperature variations. The effects of creep and shrinkage on crack formation are twofold. First, creep and shrinkage will lead to prestress loss. Second, creep and shrinkage can lead to a redistribution of stress from the concrete to the reinforcement. Shrinkage can cause additional tensile stresses whenever the shrinkage deformation is restrained. The restraints may be external from surroundings or internal from the presence of reinforcement. Shrinkage effects are generally small in large offshore structures. Calculation of shrinkage-induced stresses is necessary only when these relatively small stresses will be of critical importance. In other cases, the effects of creep and shrinkage can be ignored in checking decompression and crack formation by providing extra prestress or reinforcement.

Temperature deformation in massive offshore GBS platforms is generally recognized as being significant. In many cases, it has been included in checking the state of decompression and crack formation. However, where adequate provisions are made to accommodate thermal movements, uniform temperature change will not induce significant stresses.

Although the decompression state is checked against crack formation for the service load condition, cracking may occur under the rare loading condition, such as ship impact and earthquake. Past experience shows that the bond slip at the cracks will resist complete closure of the crack after the accidental load is removed. Therefore, a stress state of zero compression in concrete will not ensure complete closure of cracks after an accident. Tests show that a minimum compressive stress of 200ρ MPa (29ρ ksi) is generally required to ensure the closure of the cracks, where ρ is the tensile reinforcement ratio.

Calculation of crack width

The cracking behavior of reinforced concrete can be generally divided into two limiting states, as shown in Figure 16. In State I, concrete is uncracked until the tensile stress exceeds the tensile strength of the concrete. Once the concrete cracks, an increasing load will cause an increase in the number of cracks and a decrease in the stiffness until a stabilized crack pattern is developed. This corresponds to the State II in Figure 16. In State II, any further increase in loading will only widen the cracks, but not lead to formation of more cracks.

If cracking results from imposed deformation, such as restraint to deformation from shrinkage or temperature, the cracks usually never reach the stabilized State II

because the tensile stresses tend to decrease with reduction of stiffness. On the other hand, cracking due to external loading is likely to approach State II.

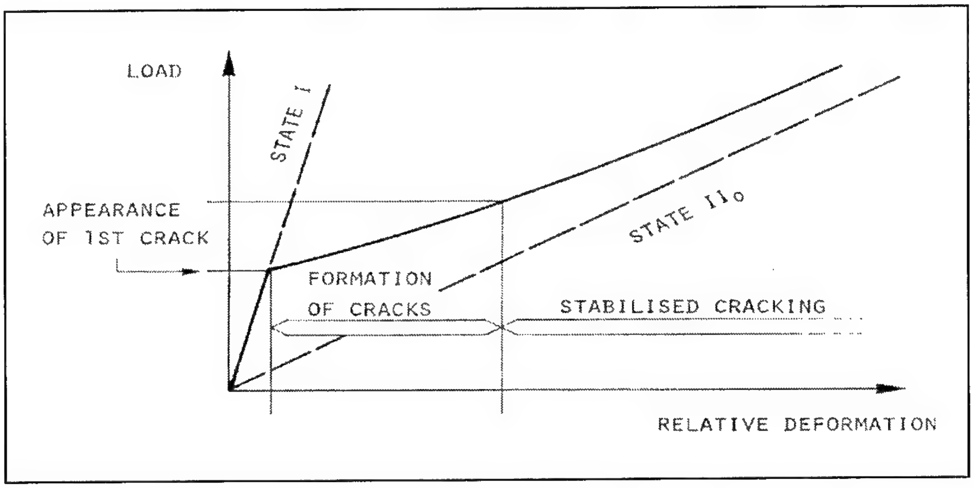


Figure 16. Load–deformation responses and two limiting states of cracking

In offshore design criteria, the crack width calculation refers to the stabilized crack pattern in State II. The calculation method is often a modified version of the CEB-FIP approach (CEB 1978). In this method, the average crack width (w_m) is equal to the average crack spacing (s_m) times the average increase in strain of the reinforcement relative to the adjacent concrete (ε_{sm}):

$$w_m = s_m \varepsilon_{sm} \tag{6}$$

The influence of shrinkage and temperature can be easily included by adding the additional strain (ε_{ca}) to the equation:

$$w_m = s_m(\varepsilon_{sm} + \varepsilon_{ca}) \tag{7}$$

The average strain increase in steel can be estimated as follows:

$$\varepsilon_{sm} = \varsigma \varepsilon_{s2}$$

$$\varsigma = \left(1 - \beta_1 \beta_2 \left[\frac{\sigma_{sr}}{\sigma_{s2}}\right]^2\right) \tag{8}$$

where ε_{s2} is the calculated strain in the steel assuming a fully cracked section and neglecting the contribution of concrete tension stiffening. Variable ς is a strain reduction factor to allow for the tension stiffening effect of the concrete. Variables and σ_{sr} are the steel stresses in a cracked section under actual load and at initial crack formation, respectively. Variable β_1 is a factor characterizing the bond quality of the steel (β_1 is 0.5 for smooth bars, otherwise is 1.0). Variable β_2 is a

factor representing the effect of load type (β_2 is 0.5 for cyclic load or long-term load, otherwise is 1.0).

The average crack spacing is established on the empirical basis as follows:

$$s_m = 2\left(c + \frac{s}{10}\right) + \kappa_1 \kappa_2 \frac{\phi}{\rho_r} \text{ (mm)} \tag{9}$$

where

c = concrete cover

s = rebar spacing

κ_1 = factor to account for the bond properties of the rebars (equals 0.4 for reinforcing bars, 0.8 for prestressing strands)

κ_2 = factor to account for distribution of tensile stress (0.125 for bending, 0.25 for axial tension)

ϕ = bar diameter

ρ_r = ratio of tensile reinforcement to the effective concrete area

Definition of the effective concrete area is necessary because there are only limited possibilities for tensile forces in the steel to be transferred into the concrete by bond within the finite distance between cracks.

Conceptual illustrations of dispersion of tensile stress into concrete and the effective concrete area used in offshore design codes are shown as Figures 17 and 18.

Due to the wide variation in crack spacings, there will also be a wide variation in crack widths. To account for this variation, offshore design codes often define the extent of cracking by the "characteristic crack width" W_k, which is defined as the crack width having approximately 5 percent fractile of exceedancy on the basis of experimental statistics. The relationship between the mean crack width and the characteristic width is as follows:

$$W_k = 1.7 w_m \tag{10}$$

When the principal stresses act at an angle with the reinforcement, the effectiveness of reinforcement in limiting crack width should be reduced. In crack width calculation, force equilibrium and strain compatibility are to be fulfilled assuming no shear transfer along the crack. Assuming that the reinforcement is laid in the x and y direction, the crack spacing $S_{r\theta}$ in the principal strain can be calculated from the crack spacing in the x-direction and the y-direction (S_{rx} and S_{ry}) as follows:

$$\frac{1}{S_{r\theta}} = \frac{\sin\theta}{S_{rx}} + \frac{\cos\theta}{S_{ry}} \tag{11}$$

where θ is the angle between the principal strain and the x-direction, as shown in Figure 19.

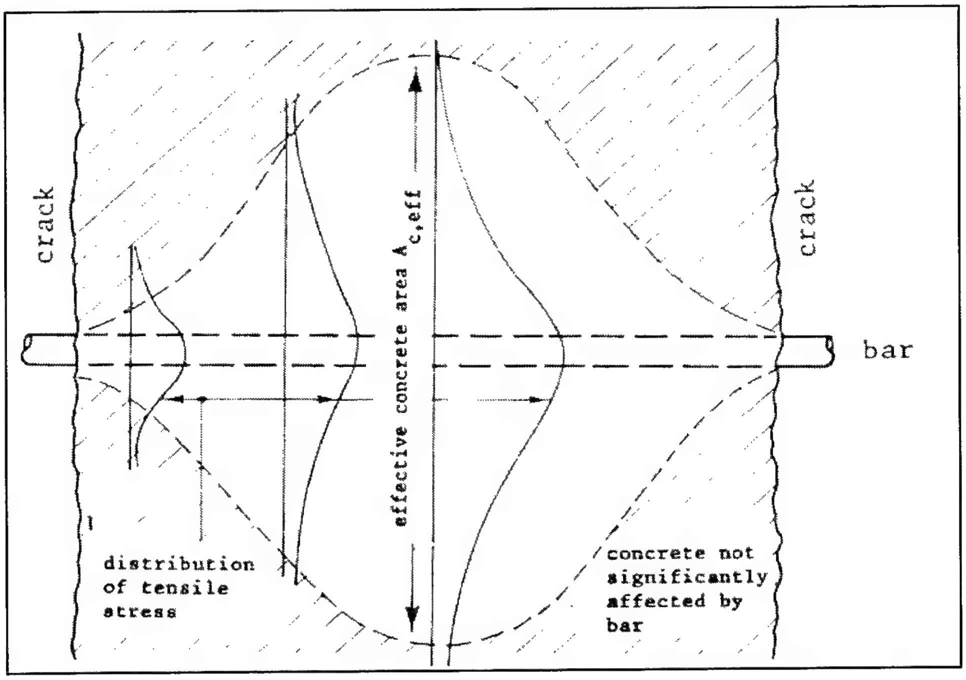


Figure 17. Conceptual illustration of transfer of tensile stress from steel bar into concrete between cracks

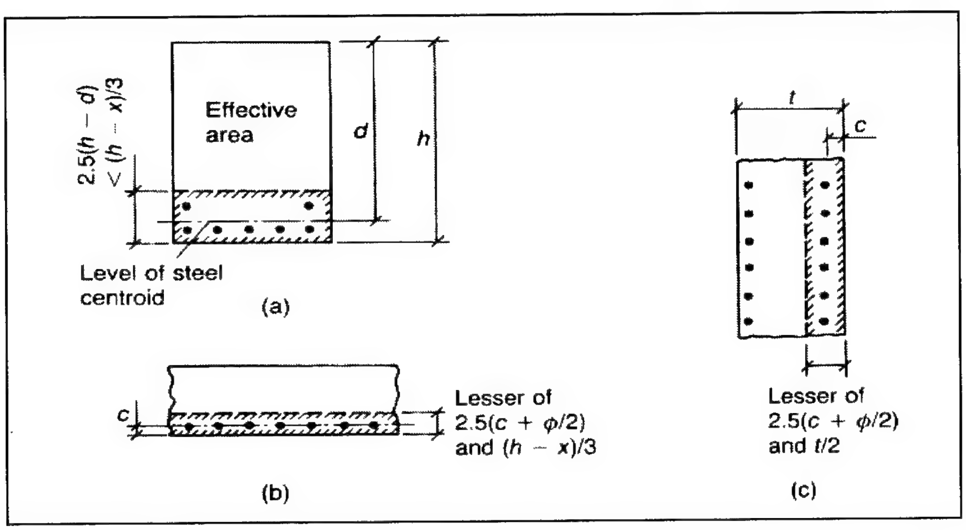


Figure 18. Effective tension area: (a) beam, (b) slab, (c) member in tension (note: where ϕ is the average diameter of steel bars, x is the depth of the compression zone prior to cracking)

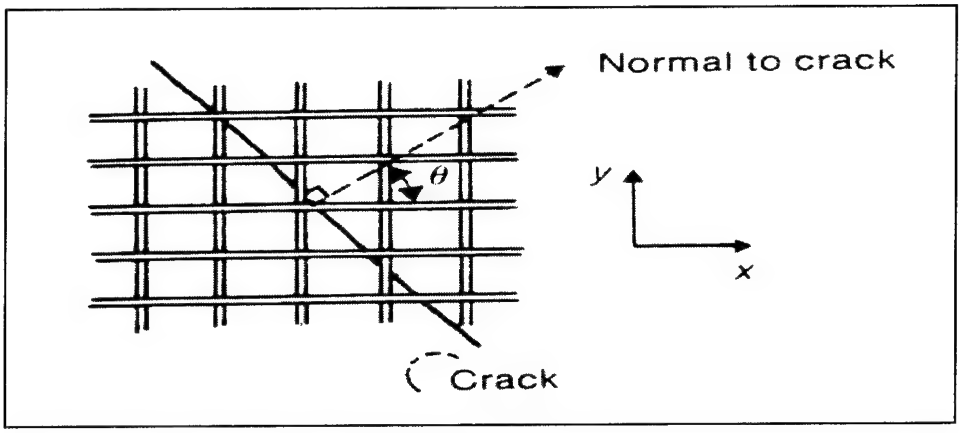


Figure 19. Crack Inclined to reinforcement

The method for calculation of the crack width in offshore design criteria is by no means unique. Several methods for checking the crack width are currently being proposed by various design codes and specifications. For example, the ACI 318 Building Code does not require checking of the crack width in prestressed concrete. Its crack control requirements for reinforced concrete are based on the Gergeley-Lutz approach. Unlike the offshore GBS design criteria, the Gergeley-Lutz approach uses a simple empirical equation that relates maximum crack width to three variables—steel stress at the crack, concrete cover, and area of concrete around each bar. The Gergeley-Lutz equation is based primarily on test results of beams and can be extended reliably for checking one-way slabs. However, experiments show that flexural cracking behavior of concrete slab in two-way action is significantly different from that in one-way members, and the Gergeley-Lutz approach often seriously underestimates the crack widths developed in two-way slab (Nawy 1992).

Mathematical prediction of the crack width is a very approximate process. Nevertheless, the method used for calculation of the crack width can have a significant impact on the resulting design. In some cases, the reinforcement requirement to control cracking is up to 30 percent higher than that required for satisfying the ULS. Therefore, considerable engineering judgment must be exercised to specify the allowable crack width limitations.

In summary, the crack control design should be based upon experience and observation of existing structures performing in similar environments. For design of offshore structures, the general approach of using the allowable stress criteria is recommended for crack control. The crack width criteria are recommended only as a check in special areas, such as the splash zone, critical structural members susceptible to fatigue damage, or structural members that do not meet the allowable stress criteria.

Ultimate Limit State Criteria

Prestressed concrete structures must be designed to preserve adequate safety against failure. The modes of possible failure of offshore structures at the ULS include the following items:

a. Loss of overall equilibrium.

b. Failure of critical sections of structural components.

c. Instability resulting from large deformation.

d. Excessive creep or plastic deformation.

The ULS design of offshore concrete structures usually follows the governing standard code, such as DNV (1997), CSA (1994) CEB/FIP Model Code (CEB 1990), and American Petroleum Institute recommended practice 2A (API 1997). In addition, general recommendations for design of offshore structures are provided in various professional technical committee reports, such as in the ACI "Guide for the Design and Construction of Fixed Offshore Concrete Structures" (ACI 1984).

A typical offshore GBS platform (Figure 20a) consists mainly of plates and shells and is subjected to various complex loads during different load stages. Therefore, structural analysis is commonly performed with the finite element method (Figure 20b). For all the critical load combinations, sectional forces (axial loads, bending moments, and shears) are calculated at various critical locations of the structure. At every location, the sectional resistance is checked against the loading demand, taking account of nonlinear response of concrete. The loading demand on a typical element can be expressed in terms of eight stress resultants (see Figure 20c): three membrane forces (N_x, N_y, N_{xy}), three bending moments (M_x, M_y, T_{xy}), and two out-of-plane transverse shear forces (V_{xz} and V_{yz}).

Most offshore design codes and specifications allow three design methods to be used in the sectional strength design:

a. Principle of superposition of component resistance.

b. Rational analytical models using a failure criterion for the multi-axial state of stress in the concrete, e.g., the Modified Compression Field Theory.

c. Variable strut-tie model.

The traditional design has been based upon the principle of superposition. It is a simplified equilibrium approach that decouples the effects of transverse shear from those of membrane forces and moments. The design of shell elements subjected to the three membrane forces and three bending moments is a generalization of the strain compatibility approach. The design for the transverse shear is based upon the equivalent beam assumption and empirical rules.

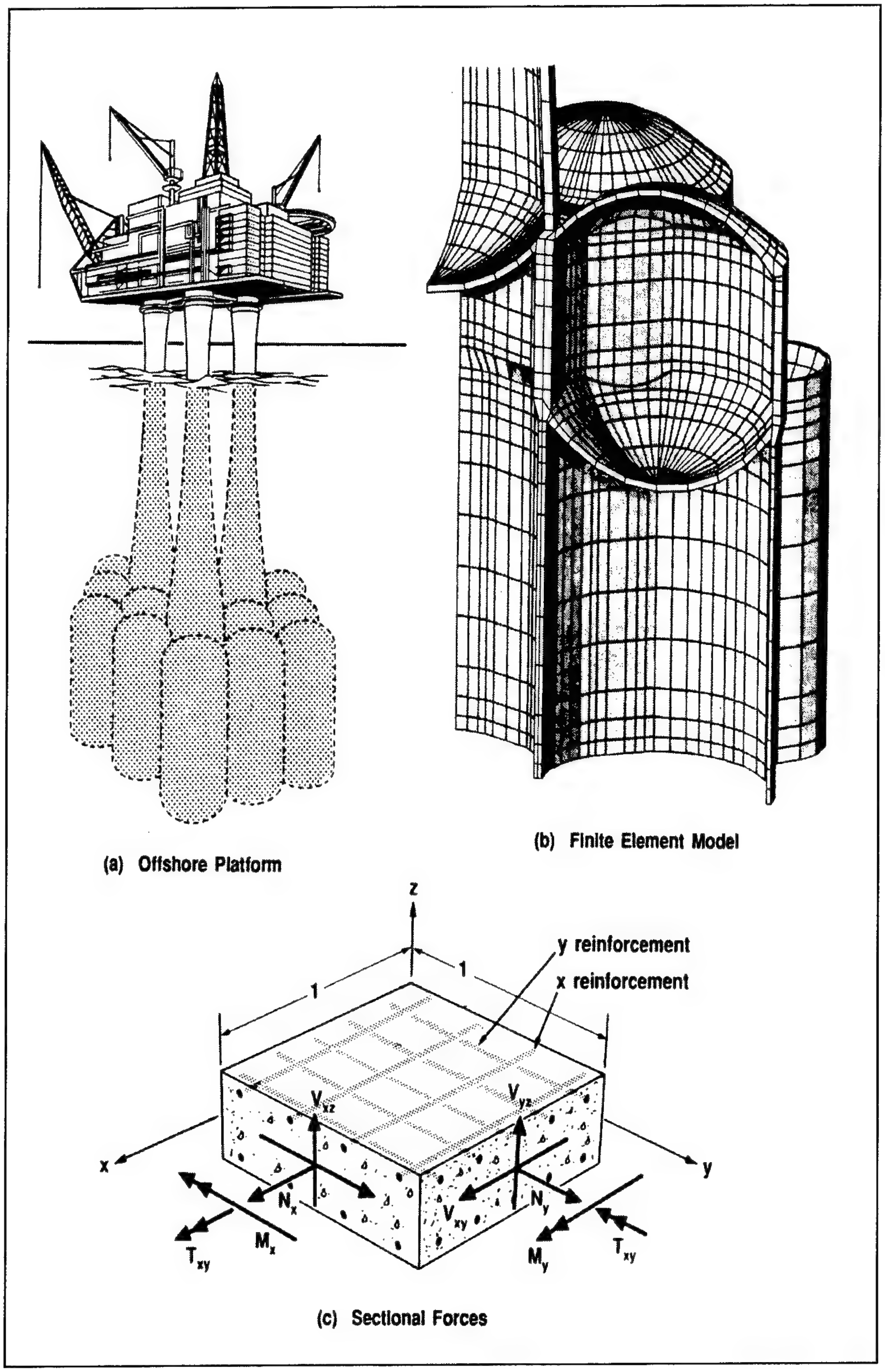


Figure 20. Analysis models for offshore GBS platforms

The multi-axial stress field method is intended to address some deficiencies with the traditional design method. The method finds a compatible strain-stress field that balances the sectional forces by an iterative process. Equilibrium is then

established at the crack surface to determine the ultimate strength, ductility, and failure mode of each individual element. Due to its analytical complexity, the stress field analysis must rely on a computer program.

The strut-tie model is a very effective tool for investigating the discontinuity zone. It can also be used, although less frequently, to analyze a large portion of the structure.

In accordance with the primary load types acting on a member, one-dimensional structural members can be categorized as beams, columns, struts, and ties. In a similar way, shell elements may be divided as (1) shell subjected to membrane forces only, (2) shell subjected to membrane forces and bending, and (3) shell subjected to membrane forces, bending, and transverse shear.

The structural strength design of shell elements to a great extend depends on the primary types of loading to which the shell elements are subjected. In the following, only the simplified equilibrium approach and the stress field method will be discussed.

Shell subjected to membrane forces only

A considerable portion of the plate and shell in an offshore platform is subjected to membrane forces (N_x, N_y, N_{xy}). In such cases, analysis can be conducted considering only the in-plane stress and strains, assuming the stresses and strains are constant through the thickness.

Many offshore design codes and specifications allow a simplified equilibrium approach to design membrane shell element without considerations of the tensile strength of concrete and deformation. The simplified design approach assumes that reinforcing steel is distributed symmetrically about the midsurface of the shell and at least one principal membrane force is tensile. The reinforcement along the x- and y-axes is designed to carry two sets of forces F_x and F_y, respectively. The forces, F_x and F_y are to be determined from equilibrium as follows (Figure 21):

$$
\begin{aligned}
F_x &= N_x + \left|N_{xy}\right| \cot\theta \\
F_y &= N_y + \left|N_{xy}\right| \tan\theta
\end{aligned}
\tag{12}
$$

In addition, the compressive strength of concrete in the principal direction at an angle θ to the x-axis should be checked against compressive failure as follows:

$$
\sigma_c = \frac{\left|N_{xy}\right|}{h \sin\theta \cos\theta} \leq f_{c2} \tag{13}
$$

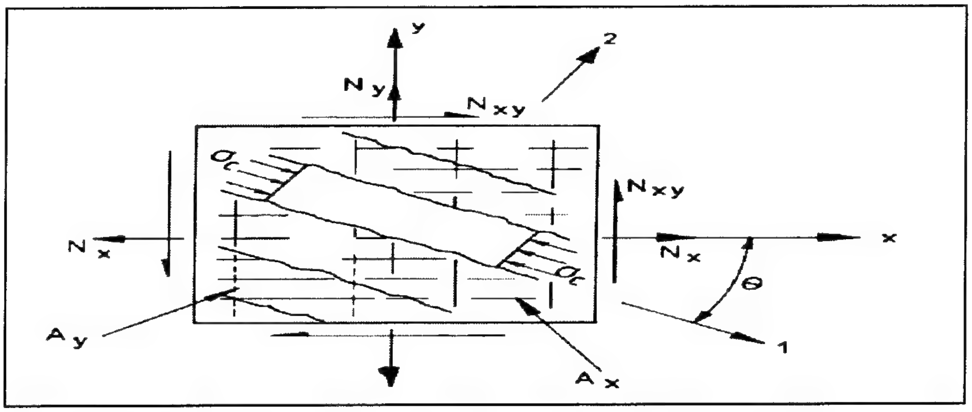


Figure 21. Sectional forces and symbols of a membrane shell element

where f_{c2} is compressive strength of the concrete, considering the potential reduction of the strength in the state of biaxial compression/tension, and h is the thickness of the element.

The signs of the membrane forces and stress σ_c are shown in Figure 21. When the membrane force N_x is compressive (negative in value) and exceeds $|N_{xy}|\cot\theta$, F_x is negative and only minimum reinforcement is required in the x-direction. Then, the reinforcement in the y-direction should be designed for F_y as follows:

$$F_y = N_y + \frac{N_{yx}^2}{|N_x|} \tag{14}$$

When both N_x and N_y forces are compressive and $N_x N_y \geq N_{yx}^2$, only minimum reinforcement will be required. From the above equilibrium equations, it is easy to show that prestressing can be used effectively to increase the resistance in membrane shear force.

While the above equilibrium approach is very simple for hand calculation and generally satisfactory for design of membrane elements, it is recognized that more refined analysis can be conducted just as easily with the assistance of proper computer programs. Therefore, many current offshore design codes also recommend more rigorous design methods on the basis of the Modified Compression Field Theory (Collins, Adebar, and Krishchner 1989) as an alternative to the simplified equilibrium method. The refined analysis method determines the response of a cracked element by separately considering the response of reinforcement and the response of cracked concrete. The average values of stresses and strains between cracks and at cracks are used in the analysis. The average strains are first determined from the sectional forces by an iterative analysis. The iterative analysis is based upon the assumption that a valid strain state is one in which the stress resultants found by integrating the resulting stresses balance the applied sectional forces.

Once a strain field is obtained, the stresses in the reinforcement and concrete are calculated with the stress-strain relationships. Equilibrium is established in the directions perpendicular to a crack and along it, as shown in Figure 22.

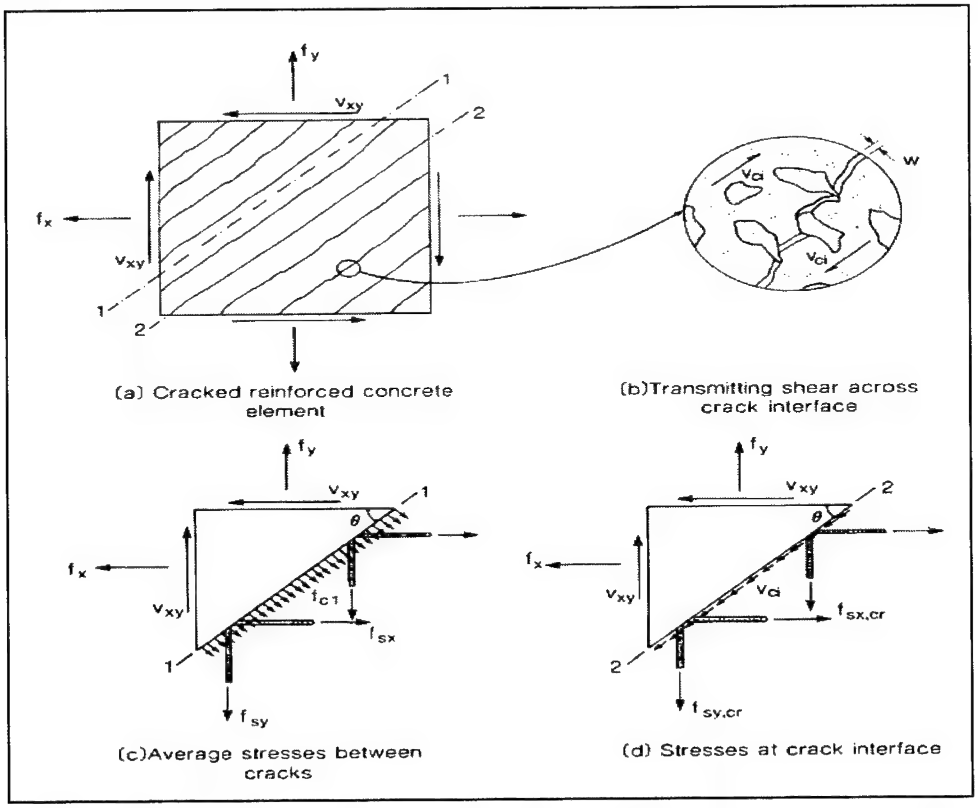


Figure 22. Assumptions of stress distributions between cracks and at cracks

The principal compressive stress between cracks has to be resisted by the compressive strength of the concrete with proper consideration of the principal tensile stress in the perpendicular direction. In addition, the shear transfer across the cracks has to be checked, taking into account the mean crack width and the steel stresses at the cracks. The crack width is to be determined according to the calculation method discussed in the previous section (Crack Control Criteria). Research has found, however, that the crack width has only a minor influence on the shear capacity until the reinforcement across the crack yields.

Shell subjected to membrane forces and bending

When a shell element is subjected to both membrane forces (N_x, N_y, N_{xy}) and bending moments (M_x, M_y, T_{xy}), analysis can be conducted considering only the in-plane stress and strains, assuming the stresses and strains vary linearly through the thickness.

Most offshore design codes and specifications allow the use of the simplified equilibrium methods. The design method is an extension of the equilibrium

approach for membrane elements. The basic procedure is to divide the shell element into a sandwich consisting of two outer layers and a central zone. The membrane force and moments are resolved into statically equivalent "membrane" forces on the outer layers, as shown in Figure 23. Then, equilibrium can be established to calculate the reinforcement in the top and bottom layers.

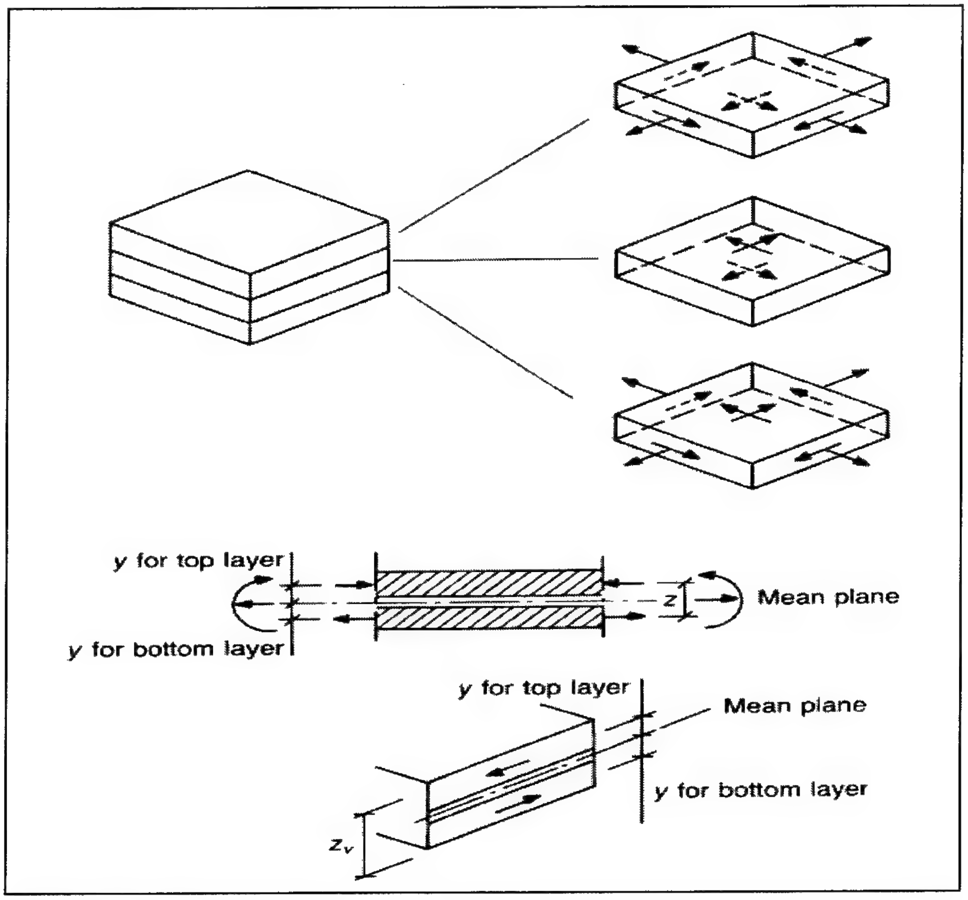


Figure 23. Simplified equilibrium approach (sectional forces are resolved into membrane forces in sandwich layers)

Many modern offshore design codes and specifications also recommend a more sophisticated "stress field" approach as an alternative to the equilibrium method. The stress field approach subdivides the element into many layers, as shown in Figure 24. Within each layer, only the in-plane stresses and strains are considered, and the Modified Compression Field Theory is applied in the same way as discussed in the previous section with regard to the membrane elements. Again, the strains are calculated from the sectional forces by an iterative analysis. Then, stresses are calculated according to the multi-axial stress-strain relationships. It is assumed that a valid strain state is one in which the stress resultants found by integrating the resulting stresses balance the applied sectional forces.

Computer programs have been developed to calculate the stresses, strains, and sectional resistance using the layered shell approach (Fiskvatin and Grosch 1982, Kirschner and Collins 1986). Figure 25 illustrates the experimental results and

analytical predictions of response of shell elements subjected to various combinations of membrane shears and bending moments. It is apparent that the flexural bending strength of the shell element is strongly coupled with the in-place shear forces.

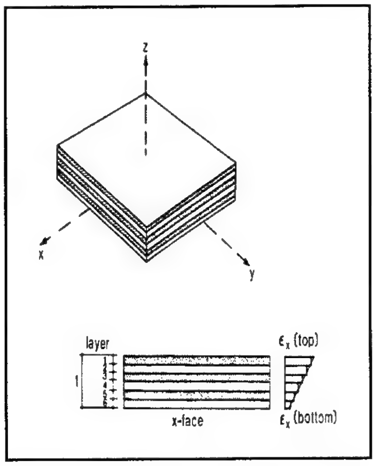


Figure 24. Layered shell element

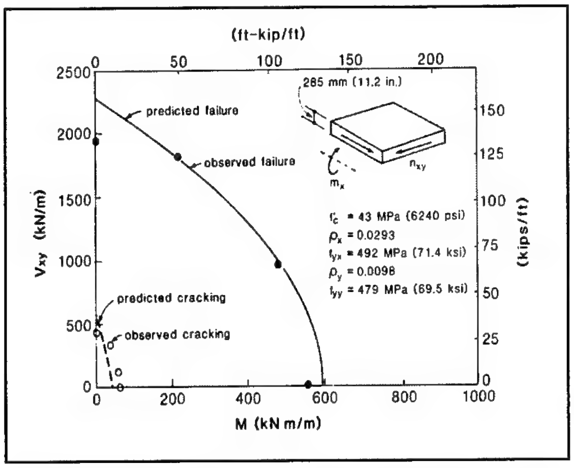


Figure 25. Shear-moment interaction diagram

This interactive response of the shell element under the combined load actions is especially important for offshore platforms in deep sea. Unlike building structures where significant membrane shear and bending moment rarely occur at the same location, shafts and hulls of offshore structures are frequently subjected to large cyclic membrane shear and out-of-plane moments. For a given moment, high membrane shears not only precipitate the initiation of cracking in the shell, but also substantially reduce the ultimate strength. Therefore, it is not acceptable to calculate the flexural strength of structural components without considering the in-plane membrane forces.

Shell subjected to membrane forces, bending, and transverse shear

In the design of offshore GBS platforms, it is often found that significant shear forces and bending moments have to be transferred at intersections between structural elements, resulting in very high membrane forces, moments, and shears in the adjoining members. Adequate strengths at these intersections and the adjoining members are critical to the integrity of the structure, as failure of these highly stressed members can lead to sudden and disastrous consequences. The consequence of the shear failure in offshore structures was dramatically demonstrated by the August 1991 collapse of the $300-million Sleipner A in Norway, making it the most costly shear-failure case in history.

As a result, most of the recent offshore design codes impose stringent requirements on shear-resistance design due to the severe environments and consequence of shear failure. The most common shear design requirements are as follows:

a. Bending moment and deformation loads must be properly considered in shear capacity calculation.

b. In slabs and walls, shear transfer should be checked in all directions on the basis of elastic analysis, unless a thorough study documenting redistribution of forces confirms the validity of an alternative approach.

c. All the prestressing tendons should be confined between layers of transverse reinforcing steel in slabs and walls. Curved prestressing tendons shall be adequately confined by lateral reinforcement.

d. In sections having a combination of high compressive stresses and high shear forces, the main reinforcement should be confined with through-thickness stirrups in an amount not less than 0.4 percent of the concrete area.

e. All load-bearing embedments must be properly anchored into the concrete with reinforcement (additional stirrups) to transfer the tension force. This requirement addresses the potential pull-out or punching-through type of failure where only part of the thickness of the structural element is activated.

f. Tensile forces perpendicular to the shell plane, if any, must be taken by extra reinforcement properly anchored in the concrete.

g. For structural elements directly exposed to water pressure, the effects of water pressure penetrating into a crack should be taken into account in determining the magnitude of the sectional forces.

The design of sectional shear strength of reinforced or prestressed concrete under various load combinations has been one of the most controversial subjects. During the last four decades, various theories and design methods have been proposed. Most offshore design codes allow several methods to be used for determination of the shear resistance, such as the equivalent beam method and the multi-axial stress field method.

The traditional shear design practice has been based on the principle of superposition of component resistances, or so-called "equivalent beam" method. In this method, evaluation of the transverse shear resistance is separated from evaluation of the flexural strength and in-plane shear strength. The evaluation of the sectional shear strength of shell elements has been traditionally based upon empirical procedures developed for one-dimensional members.

It should be recognized that, for one-dimensional elements, the axial load, transverse shear, and longitudinal reinforcement are all parallel. For the shell and plate, the loading condition can be much more complex. For example, considering an equivalent beam strip taken out of a shell element in Figure 26, the beam strip may be subjected to the principal transverse shear on the unit width without axial tension. But the "beam" would also be subjected to axial tension on its "side faces"

across the width. The influence of the tension on the shear strength cannot be accounted for in the equivalent beam design procedure.

The shear design procedure on the basis of the superposition principle is to take an equivalent beam strip in a shell element, as shown in Figure 26, and calculate the transverse shear force and the axial force on unit width of the beam strip as follows:

$$\begin{aligned} V &= V_x \cos\alpha + V_y \sin\alpha \\ N &= N_x \cos^2\alpha + N_y \sin^2\alpha + 2N_{xy} \sin\alpha\cos\alpha \end{aligned} \tag{15}$$

where α is the angle between the axis of the beam strip and the x axis.

The shear and axial force so obtained can be applied directly to the conventional beam design equations, such as those stipulated by ACI 318 Building Code.

Past experience has proven that the equivalent beam method is valid for shear forces combined with moderate bending moments and axial forces. However, when high bending moments and high axial forces occur simultaneously in shell elements, the equivalent beam method may give inaccurate results.

A close examination of the equivalent beam approach reveals several drawbacks with the method. In order to apply the beam shear equations to the equivalent beam strip (see Figure 26), the forces on the side faces of each beam strip are neglected. The influence of the orientation of the reinforcement is also neglected. As a result, the method cannot accurately predict the influences of membrane shear and axial force on the transverse shear strength of the structures.

When a shell element is subjected to a combination of high shear, bending moment, and axial forces, it is justifiable to use more refined shear design methods. For example, CSA S474 and DNV codes recommend the use of the analytical models based upon the Modified Compression Field Theory.

The analytical model based on the Modified Compression Field Theory is basically an extension into three dimensions from the model for membrane shell, as discussed in the previous sections. The stress field model has been incorporated into computer programs (Collins, Adebar, Kirschner 1989). Figure 27 illustrates the observed test results and predicted responses of a number of shell elements subjected to combined membrane shear, transverse shear, and bending moment. The analysis results of the stress field model are compared with those of the equivalent beam method in the figure. It is apparent that the equivalent beam method fails to predict the interaction between the membrane shear and transverse shear.

Tests also show that the equivalent beam method may become considerably unconservative when very high membrane compression coexists with significant transverse shear, and may become excessively conservative when high membrane tension exists.

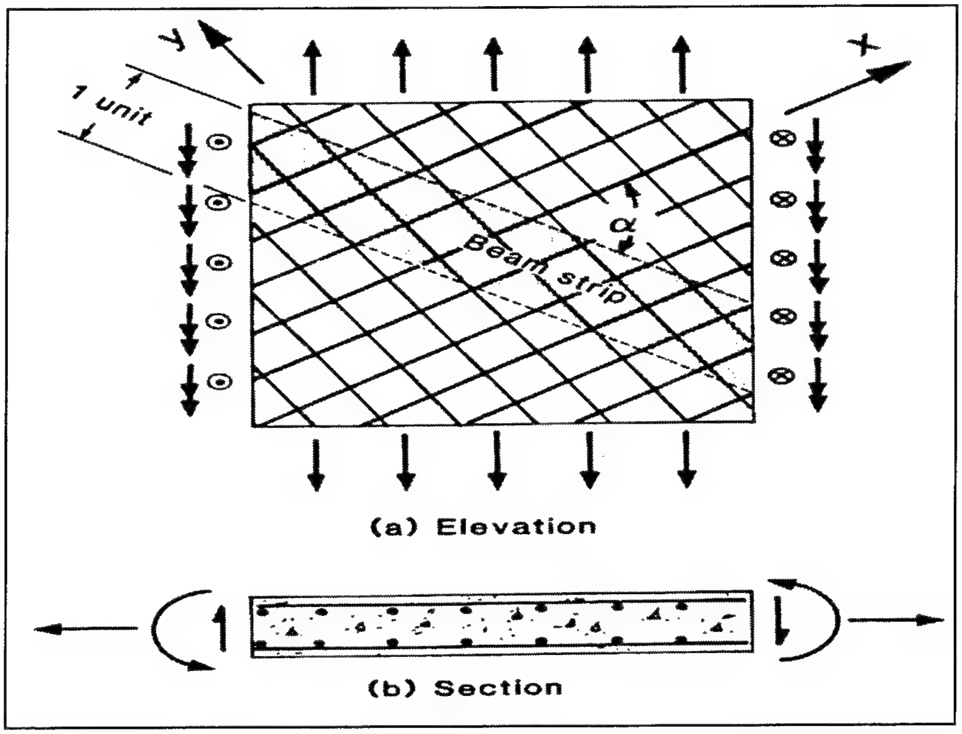


Figure 26. Equivalent beam model as applied to shell elements

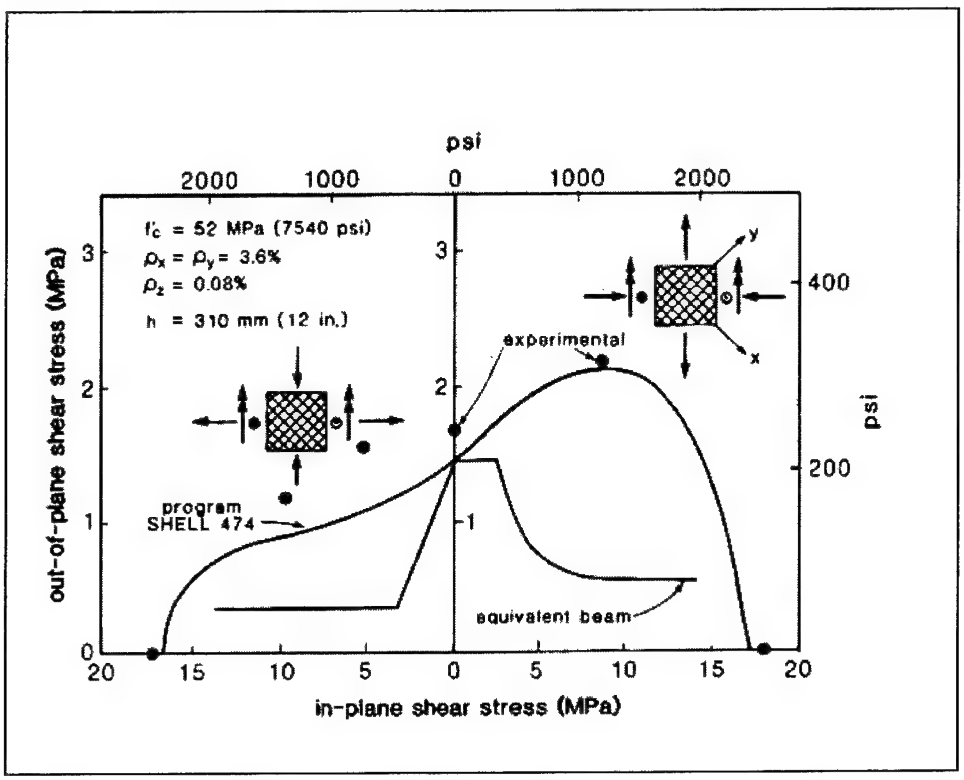


Figure 27. Interaction among membrane shear, transverse shear, and moment

It is noted that the ASSHTO Bridge Specification (LRFD) has formally adopted the shear design method based on the Modified Compression Field Theory. The ACI 318 Committee is in the process of adopting the stress field method in lieu of the traditional empirical shear design procedure.

Stress conditions at intermediate prestressing anchorage

The local stress conditions at prestressing anchorages, in the anchorage zones, between adjacent anchorages (e.g., in zones of multiple anchorages), and beyond anchorages in the direction away from the tendons, are very complex. Offshore design specifications commonly require detailed investigation of the stress conditions in those discontinuity zones under maximum loads (usually initial prestressing). More restrictive crack width criteria are usually imposed in the anchorage zones to prevent bursting at these critical locations.

Almost all the prestressing in offshore GBS is posttensioning. Construction of these gigantic structures requires numerous intermediate anchorages as the structure is being built in stages. As a typical intermediate anchorage curves away from the longitudinal member, as shown in Figure 28, the prestressing force will produce high transverse deviation force, tension force along the member, and splitting shear at the anchorage.

The transverse deviation force exerts radial pressure to spall the concrete cover. This force must be calculated and fully resisted by tieback stirrups anchored into the compression zone of the concrete.

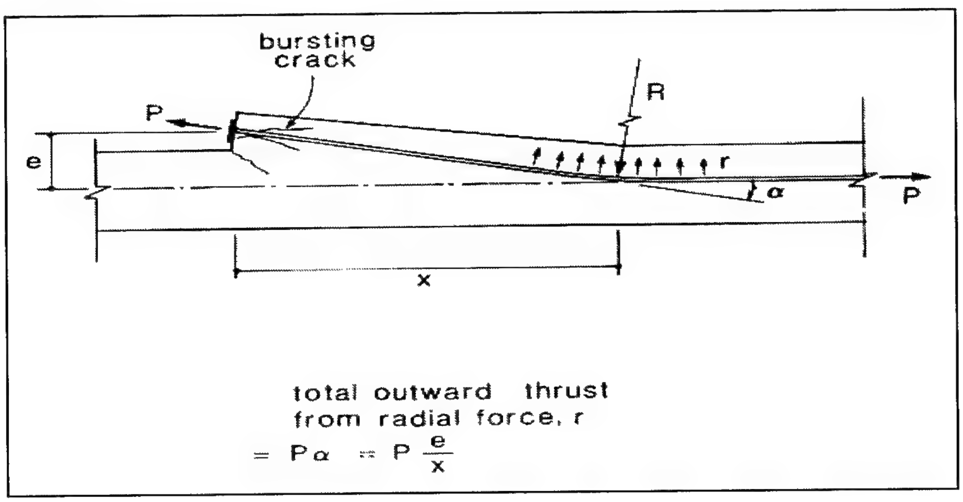


Figure 28. Slitting cracks and radial forces at immediate anchorage

When the prestress is applied at the intermediate anchorage, the concrete in front of the anchorage is compressed, imposing a tension force to the concrete behind the anchor. The tension force must be resisted by adequate reinforcing steel. In the absence of detailed strain compatibility analysis, a common rule is that the capacity of the added mild steel should be equal, at working stress, to one half the

initial tendon force. These reinforcing bars should run far enough to develop their ultimate tension, and their termination should be staggered if the member is subjected to cyclic loads.

Lateral shear is critical at intermediate anchorages due to the bolster. When groups of tendons are anchored at the same bolster, the anchorage force tends to shear off the bolster. Therefore, bolsters at intermediate anchorages should be well anchored by stirrups and should be tied laterally by reinforcing steel in the shell. Multiple anchorages are preferably staggered.

Fatigue Limit State Criteria

Offshore GBS platforms are subjected to millions of cycles of wave and current action over their entire service life span. In addition, these structures are occasionally subjected to high-magnitude cyclic loads such as breaking storm waves. The objective of the fatigue limit state design is to establish an acceptable safety margin against these cyclic loadings of variable magnitudes within the service life of the structures. The fatigue analysis of offshore GBS platforms is commonly based upon a design life of 50 years. In the fatigue analysis, the load factor is taken as 1.0. The modulus ratio of steel to concrete, E_s/E_c, is commonly taken as 10 in order to reflect the degradation of concrete stiffness under cyclic loading. All analyses should be based upon transformed section properties according to elastic theory.

Fatigue design methods

In design of offshore concrete structures, there are two distinctive design methods available for evaluation of structural components against fatigue failure: the stress limit control, and the comprehensive fatigue analysis.

The stress limitation control is a simplified design method that evaluates the fatigue strength on the basis of a set of allowable stress criteria for concrete, and for reinforcing and prestressing steel. The design method is based upon the assumption that a structural element is safe against fatigue failure if certain stress conditions are satisfied. The typical stress criteria in fatigue evaluation of offshore structure are similar to the ACI 357R-78 recommendations, as given below:

a. Maximum stress range in reinforcing and prestressing steel is less than 140 MPa. If reinforcement is bent or welded, the maximum allowable stress is 70 MPa.

b. No membrane tensile stress in concrete.

c. Maximum flexural tensile stress in concrete is less than 1.4 MPa.

d. Maximum compressive stress in concrete is less than $0.5 f_c'$.

e. If maximum shear exceeds the allowable shear stress in concrete, or if the cyclic excursions exceed 50 percent allowable shear in concrete, then all the shear forces should be taken by stirrups. Calculation of the allowable shear in concrete may account for the favorable effect of prestressing.

In many offshore design specifications, additional requirements are often imposed. For example, if lap splices of reinforcement or pretensioning anchorage are subjected to cyclic tensile stresses greater than 50 percent of the allowable static stresses, the lap length or prestressing development length should be increased by 50 percent.

If the established threshold values of the stress ranges are exceeded, or where fatigue resistance is likely to be a serious problem (which occur for only a few members in a structure), the comprehensive fatigue analysis should then be carried out.

The comprehensive fatigue analysis is based on the theory of linear cumulative damage. The design method requires that the long-term distribution of wind, wave, and current forces upon offshore structures be established and subdivided into a number of wave blocks in the form of a histogram. Then, the dynamic response of the structure is analyzed for each wave block, including appropriate dynamic amplification.

On the basis of the analysis results, total cumulative fatigue damage under the entire variable loading spectrum can be determined in accordance with the linear theory of cumulative damage, i.e., Miner-Palmgren hypothesis together with the Wöhler S-log N curves. The basic assumption of the theory implies that the long-term distribution of stress range can be represented by a stress histogram consisting of a number of constant amplitude stress range blocks, each with the appropriate number of stress repetitions:

$$\sum_{i=1}^{j} \frac{n_i}{N_i} \leq \eta \tag{16}$$

where

j = total number of load blocks considered

n_i = actual number of load cycles for load block i

N_i = number of load cycles causing failure if load block i acts alone

The above theory can be applied to evaluate fatigue strengths of concrete, reinforcing and prestressing steel, and the bond between the steel and concrete under an assumed cyclic loading histogram, provided that appropriate S-log N curves are applied and an appropriate Minor's sum η is used. In summarizing the early research results, Waagaard (1977) showed that a Miner's sum η of 0.2 to 0.5 is

appropriate for predicting the cumulative usage capacity of concrete in offshore environments.

The comprehensive fatigue analysis approach is substantially more complex than the stress limitation method. Its applications in practical design are, therefore, limited to special structures or structural components for which fatigue is likely to be a serious problem. Furthermore, the fatigue analysis method itself is empirical in nature. It provides merely lower bounds of widely scattered empirical data. The analysis results do not reflect the significant effects of sequence of loading, random variations in stress ranges, and the beneficial effects of rest periods.

Fatigue characteristics of basic materials

Fatigue failure of concrete is caused primarily by progressive internal microcracking, although external surface cracking can often be observed long before actual fatigue failure. The microcracks usually initiate at the aggregate-to-paste interface, and spread around the aggregates into the concrete matrix. Intensive development of internal cracking prior to failure causes a significant increase in both longitudinal and transverse deformation. When compressive stress reaches $0.7\ f_c^{'}$, microcracking initiates and stiffness decreases, leading to potential dynamic amplification under cyclic loading. There is evidence that Poisson's ratio also increases under the circumstances. Unlike compression failure of concrete under static loading, fatigue failure of concrete in compression is ductile in nature. The local concentration of compressive stress under repeated loading will be relieved and redistributed prior to rupture.

Tensile cracking is initiated when the tensile strength of the concrete is exceeded, either by excessive stress excursions into the tensile range which overcome both the prestress and the static strength of the concrete, or by repeated cycling leading to tensile fatigue of the concrete. Tests show that cyclic loading at about 50 percent of the static tensile strength of the concrete can cause fatigue cracking.

The fatigue strengths of concrete and steel, as well as the bond strength under a single amplitude of loading, are usually determined in accordance with the following two rules:

a. The Wöhler S-Log N design curves.

b. The modified Goodman diagram for concrete under a given number of loading cycles (usually 1.0 to 3.0×10^6 cycles).

The Wöhler S-Log N curves reflect dependence of the fatigue strength on the stress range, material properties, and environmental conditions. The curves can be generally represented as

$$\log N = C_1\left(\frac{1-\sigma_{\max}/f_{rd}}{1-\sigma_{\min}/f_{rd}}\right) \tag{17}$$

where

$\sigma_{\max}, \sigma_{\max}$ = largest and smallest compressive stress (calculated as mean stress within each stress block, for tensile stress, $\sigma_{\min} = 0$)

f_{rd} = design strength corresponding to referenced failure mode

and C_1 is a factor dependent on the environmental conditions. For example, for concrete in air, C_1 is 1.2; for submerged concrete subjected to compress/tension stress reversal, C_1 is 0.

Figure 29 illustrates the allowable number of cycles and probable actual number of cycles at each stress range in concrete of a typical offshore platform in the North Sea.

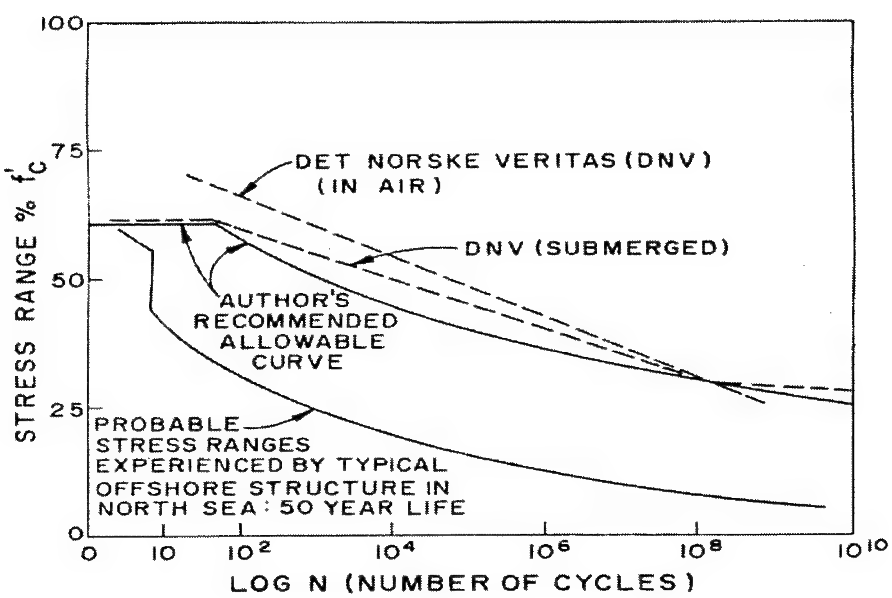
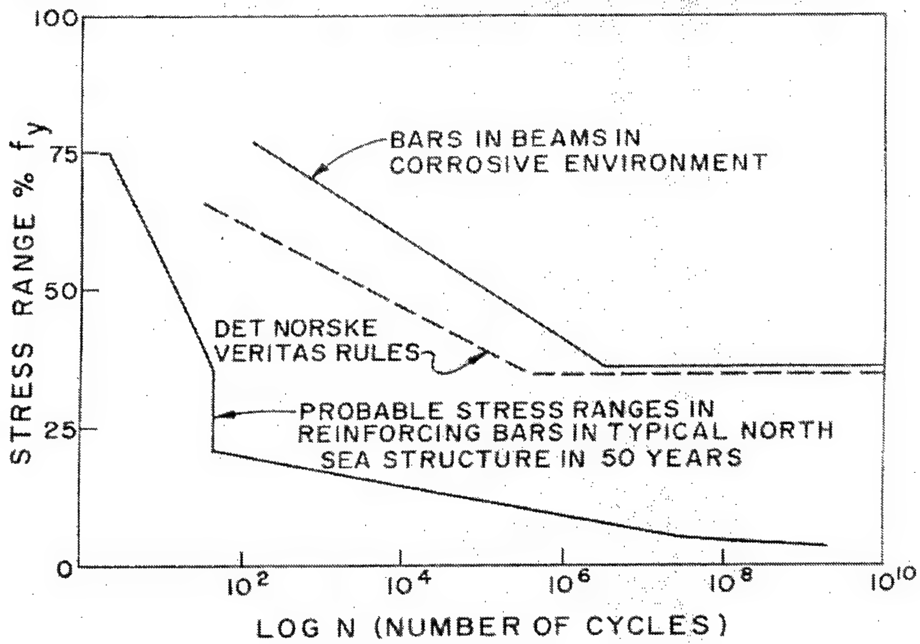


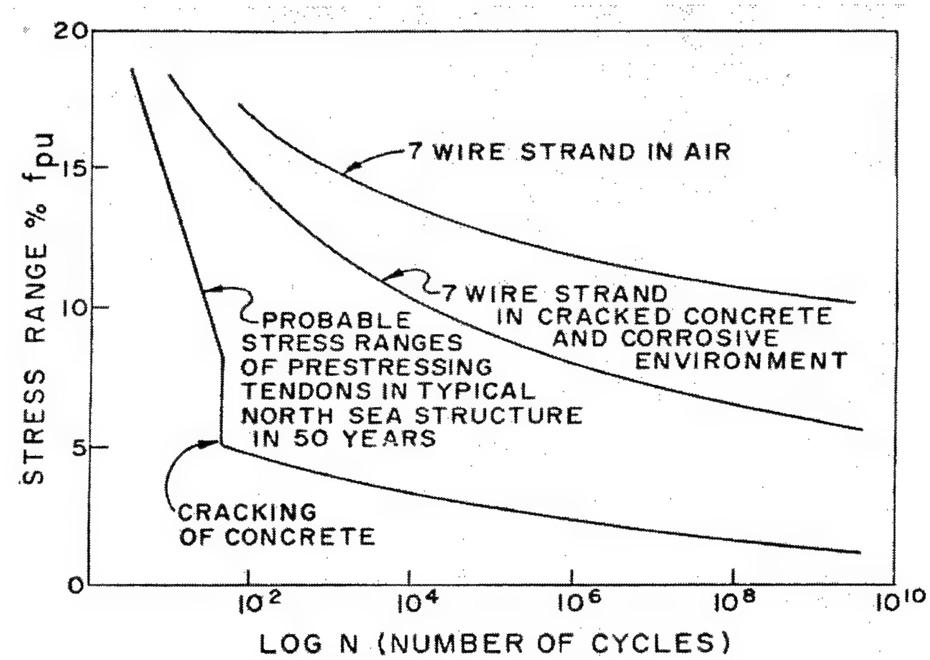
Figure 29. Wöhler curves for concrete in compression in North Sea structures

Figures 30a and 30b show similar plots for prestressing steel and reinforcing steel, respectively. Tests generally indicate that both prestressing steel and reinforcing steel have an endurance level of about 160 MPa, although a fatigue level as low as 140 MPa has been reported. These stress levels can be obtained after significant cracking has taken place.

Although concrete does suffer progressive loss of strength with increasing number of loading cycles, a comparison of the Wöhler curves developed on the



a. Curve for reinforcing steel



b. Curve for prestressing steel

Figure 30. Wöhler curves for North Sea structures

basis of laboratory tests with the probable distribution of compressive stresses during a service life of offshore structures shows extremely low probability of cumulative damage at the high-cycle end of the load spectrum. For a typical offshore concrete structure, high-cyclic fatigue has not been considered a significant problem. However, significant damage can occur at the low-cycle, high-amplitude end of the load spectrum. That is, a relatively small number of load cycles of high magnitude can cause a sizable reduction in stiffness and rapid increases in strains that leads to cracking and spalling. A combination of low-cycle, high-magnitude load and high-cycle, low-magnitude load can lead to potential failure. In addition, repeated cycling into high compressive ranges causes a substantial increase in creep and a reduction in the effective prestress.

It should be pointed out that there is a significant difference in fatigue endurance between uncracked prestressed concrete and concrete that has been precracked by a prior extreme load and subjected to continuous crack opening and closing.

When concrete is subjected to excursion into the tensile range of the concrete, but still without cracking, the allowable maximum stress range can be determined by a modified Goodman diagram, as shown in Figure 31.

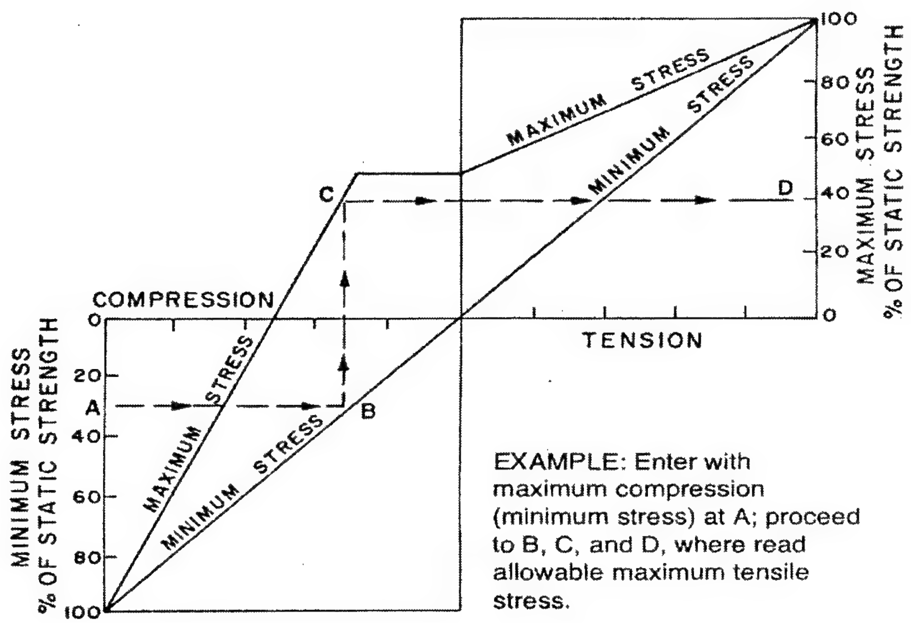


Figure 31. Modified Goodman diagram for submerged concrete subjected to fully reversing stresses, 90-percent probability of nonfailure under 3×10^6 cycles

In practice, low-cycle, high-amplitude loads may initiate cracking in concrete. Cracking may also occur due to overload, accident, construction procedure, creep, and thermal strains. Without effective prestress, the cracks can be repeatedly opened and closed by the subsequent cyclic loads at moderate magnitudes. Alternating

opening and closing of cracks causes several adverse structural responses and potentially leads to serious degradation of the strength. First, the reinforcing steel and prestress steel across cracks must pick up the tension force previously carried by the tensile capacity of concrete. Initially, "tension stiffening" of the concrete may somewhat alleviate the stress jump in steel at cracks. However, the tension stiffening is degraded rapidly under cyclic loading. This stress jump in reinforcement and prestressing steel due to cracking of concrete is indicated in the Wöhler S-Log N curves in Figures 30a and 30b, respectively.

As the cracks alternately open and close, there is also a significant jump in the concrete stress, as indicated in Figures 28 and 32. This stress increase results from the dynamic hammering effect on crack closing and also the stress amplification under decreasing stiffness. The dynamic effect of the crack closing (hammering) leads to mechanical abrasion and breaking loose of the aggregate particles. The net result is that, at cracking, there is a drastic jump in the stress range of the steel, combined with a significant increase in the maximum compression stress level in the concrete.

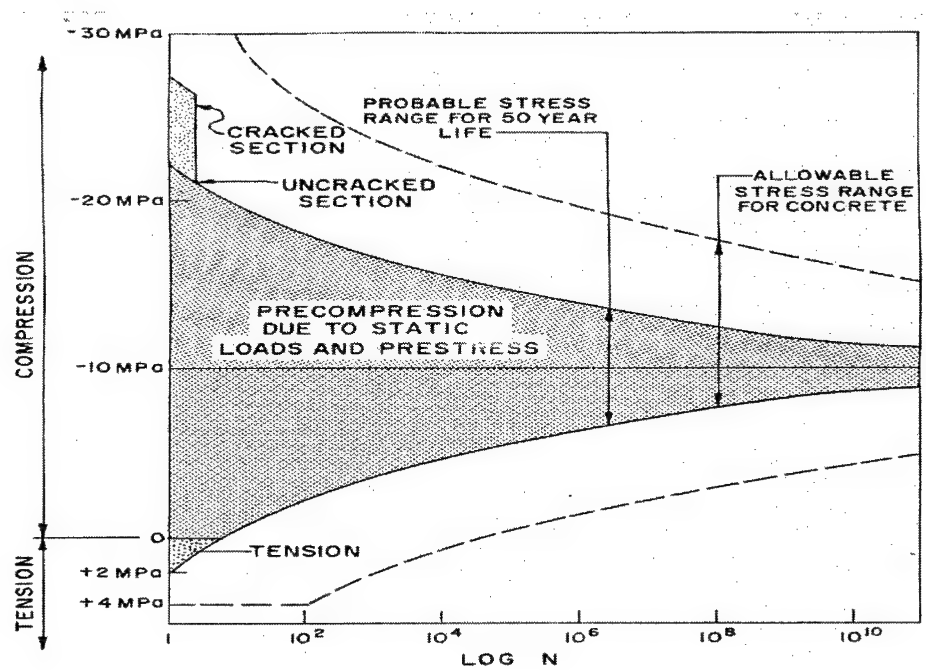


Figure 32. Probable versus allowable stress range for prestressed concrete

The repeated excursion into the tensile cracking is especially detrimental for submerged concrete. Opening and closing of the crack causes pumping of water in and out of the crack. The water in the cracks is subjected to high temporary pressures during crack closing. As the water exits the crack at high velocity, it often erodes the cement paste and loose sand grains. More importantly, the water trapped in cracks may lead to the wedging actions of hydraulic fracturing. That is, the trapped water in the crack can cause hydraulic fracture and splitting of the concrete under the instantaneous hydrostatic pressure. Numerous tests show that submersion

of concrete causes a substantial reduction of fatigue life due to the pumping and wedging of water (Balachandra 1978, Bannister 1975, Taylor and Sharp 1978, Waagaard 1977). Because submersion of concrete can accelerate the fatigue failure under cyclic loading, fatigue failure criteria for submerged concrete are more restrictive than for the concrete sections above the water.

Fatigue characteristics of prestressed concrete members

The FLS of a prestressed concrete member should be evaluated in terms of the bond strength, shear strength, and flexural strength.

Bond strength. Bond deterioration between steel and concrete is known to be a significant factor in fatigue failure of concrete structures under low-cycle, high-amplitude cyclic loading. Prestressing steel usually has a lower effective bond strength than the reinforcing steel. Under cyclic loading, therefore, progressive bond slip of prestressing tendons usually takes place first, accompanied by substantial loss of effective prestress. Although quantitative information on the effects is still lacking, it has been found that deterioration of bond strength under cyclic loadings is influenced by several factors, including

a. Depth of concrete cover.

b. Confinement of concrete around the steel.

c. Cyclic tensile stress range in steel.

d. Diameter and deformation of steel reinforcement.

e. Water pumping through cracks for submerged concrete.

f. Concrete strength.

Low-cycle, high-amplitude fatigue tests on prestressed beams (Balachandra 1978, Magurama 1978) found that fatigue failure of prestressed concrete was often preceded by cracking and loss of bond in association with splitting of concrete and yielding of reinforcing steel. The bond failure usually exhibits the two distinctive modes. The first failure mode is related to tensile splitting failure of concrete that is not fully confined, as shown in Figure 33a. The second mode is pull-out failure, which is related to shear failure of the concrete, as shown in Figure 33b.

Tensile splitting failure usually exhibits much lower bond strength than does pull-out failure. This indicates that the ultimate bond strength is highly dependent on the confinement and geometry of the concrete, as well as the loading pattern. If the geometry is such that diagonal tension cracks do not occur in the anchorage zone, the bond strength under 10^6 cycles of loads will be approximately 60 percent of the static strength. Where inclined cracks occur under repeated loading or there are significant splitting effects because of inadequate concrete cover or inadequate

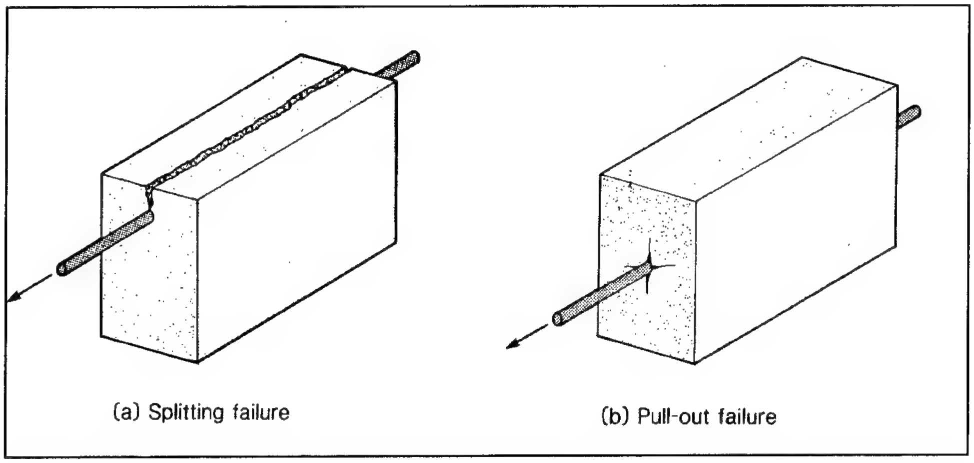


Figure 33. Two distinctive modes of bond failure

lateral reinforcement around the bars, bond strength can decrease to 40 percent of the static strength under cyclic loading.

The benefits of lateral confining steel in enhancing concrete bond strength have been frequently reported (Hawkins 1974). When concrete cover is not adequate to prevent splitting bond failure, sufficient lateral steel confinement should be provided to prevent premature bond failure under repeated cyclic loads. For high-amplitude loading, the local bond stress builds to a maximum and then deteriorates for increasing peak slips. The rate of deterioration depends on the previous maximum local bond slip. The greater the previous bond slip, the greater the reduction in bond stress. Reversals of loading on the steel bars increase the rate of bond deterioration by approximately 50 percent.

Detailed analyses of bond strength and slip under cyclic loads can be carried out with numerical bond stress-slip models (CEB 1990). The CEB-FIP model defines the relationship between an average "local bond" versus "local slip," as shown in Figure 34. The first curved part of the bond stress–slip model refers to the stage in which the ribs penetrate into the concrete, resulting in local crushing and microcracking. The horizontal level occurs only for confined concrete, representing advanced crushing and shearing off of the concrete between ribs. The descending branch represents the reduction of bond resistance due to the occurrence of splitting cracks along the bars. The last horizontal part refers to a residual bond capacity. Specific parameters of the model (τ_{max}, τ_f, s_1, s_2, and s_3) are defined for various conditions, such as confined concrete versus unconfined concrete and deformed reinforcement versus relatively smooth bars and strands. For repeated loading, a correction factor is used to account for fatigue degradation of bond strength, as shown in Figure 35.

In practice, such detailed analyses of bond slip are rarely conducted. However, in view of some bond strength-related failures of floating bridges under cyclic loading, it seems to be necessary to further investigate the bond behavior of

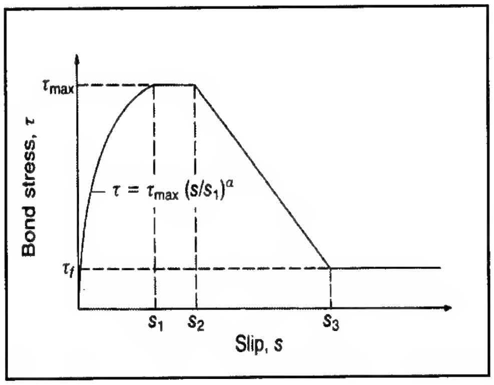


Figure 34. Bond stress-slip model

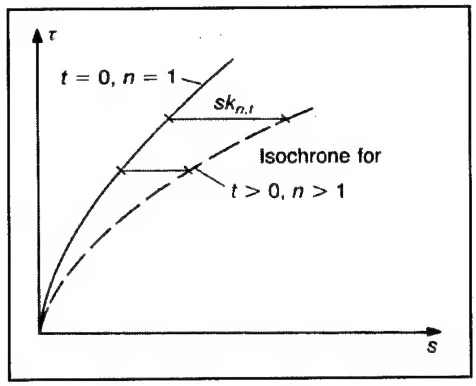


Figure 35. Correction for cyclic load

prestressed concrete and to establish a simple and reliable design criterion to guard against bond failure.

Flexural strength. On the basis of fatigue characteristics of the basic materials, calculation of the flexural fatigue strength of prestressed concrete members should be straightforward. In determining the critical fatigue stresses from the Wöhler curves, it is important to distinguish a cracked concrete section from an uncracked concrete section. Figure 36 is a typical plot of the hysteresis curves of a prestressed concrete section under high-amplitude cyclic loading.

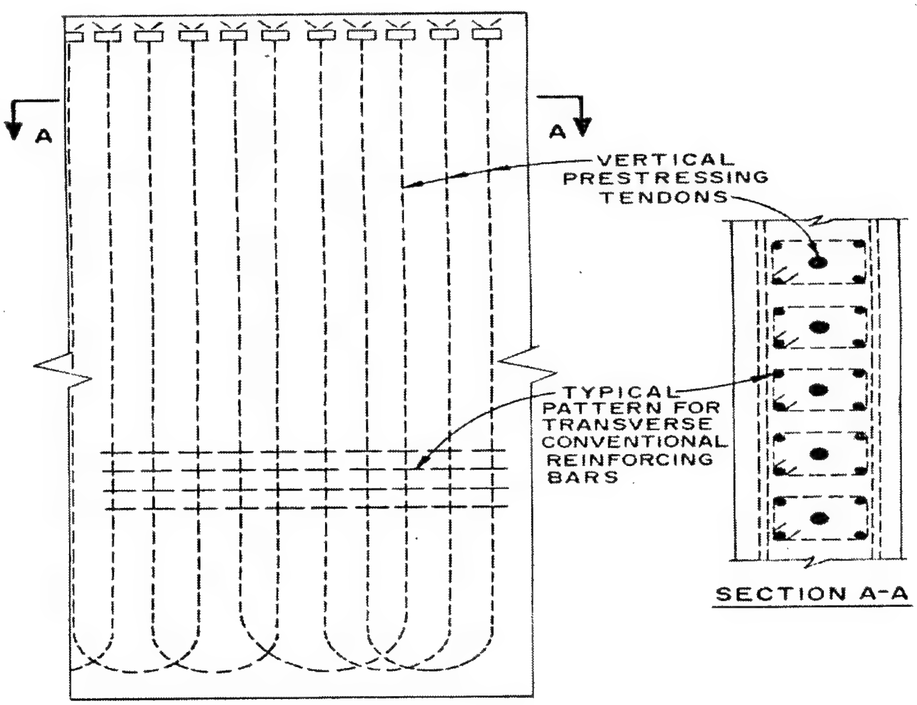


Figure 36. Hysteresis curves for prestressed concrete under high-amplitude cycling into the range of tension and cracking

This plot shows that cycling of prestressed concrete into tension and cracking will lead to a progressive degradation in the effective prestress and stiffness, resulting in wider cracks and higher steel stresses.

For prestressed concrete, flexural cracking must occur before fatigue failure becomes an important design consideration. As a rule of thumb, the fatigue flexural strength of a prestressed member is generally greater than its static flexural-cracking moment. One of the most effective ways to increase the fatigue strength is to increase the effective prestress, thereby limiting the tensile stress level in the element and controlling the occurrence of cracks. The current design practice for marine structures is to prestress concrete so that the concrete stresses rarely cycle into the tension range. If the effective prestress is sufficient to ensure structural members within the state of precompression under the service loading condition, the fatigue failure will not occur.

Shear strength. Offshore structures, both fixed and floating, resist wave forces primarily in membrane responses through cyclic in-plane shear and membrane tension/compression. Individual sections of the structural shells are also subjected to flexural bending due to hydrostatic loads or temperature differentials. Reinforced concrete beams, with or without web reinforcement, can have shear failure under cyclic loading at as little as 30 to 40 percent of the static strength. For prestressed concrete beams, shear failure under cyclic loading can be controlled by limiting stresses in concrete. The fatigue strength for web-shear cracking can be evaluated by limiting the principal tension stress in accordance with the modified Goodman diagram shown in Figure 31. The fatigue strength for flexure-shear cracking can be evaluated by limiting the extreme fiber tension stress according to the same diagram. As the magnitude of shear stress and number of the load cycles increase, the possibility of web-shear cracking occurring prior to flexure-shear cracking increases. As inclined cracks initiate, the web reinforcement will carry all the shear stresses after a number of loading cycles. The fatigue life of prestressed concrete beams after diagonal cracking should be based on predicted stress ranges in the stirrups, using the truss analogy and assuming all the shear stresses carried by the web reinforcement.

In shell elements of offshore structures, both fixed and floating, the cyclic degradation is often magnified under extreme cyclic reversing membrane shear force. The reversing shear can produce a double pattern of diagonal cracks, oriented at an angle to the grid pattern of reinforcement. The cyclic membrane shear can be viewed as a case of alternating cyclic diagonal tension and compression. When only conventional reinforcement is used, the concrete will crack under moderate to high shear forces. Hence, the reinforcement is expected to carry all the shear tension force across the cracks, and the stress range must be kept low in order to minimize the crack width and cumulative fatigue damage.

For the typical diagonal crack pattern, crack widths are enlarged as compared with cracks normal to the reinforcement at the same level of steel stress, because the reinforcement layout is not effective in controlling this type of inclined cracking (see Figure 19). Under high-intensity loading, displacement along the cracks can produce abrasion of the concrete surfaces and a rapid reduction in shear stiffness. In practice,

vertical posttensioning (see Figures 37 and 38) has proven to be valuable and practicable in preventing cracks, or at least in changing them crack toward the vertical direction where they can be more effectively resisted by the horizontal steel.

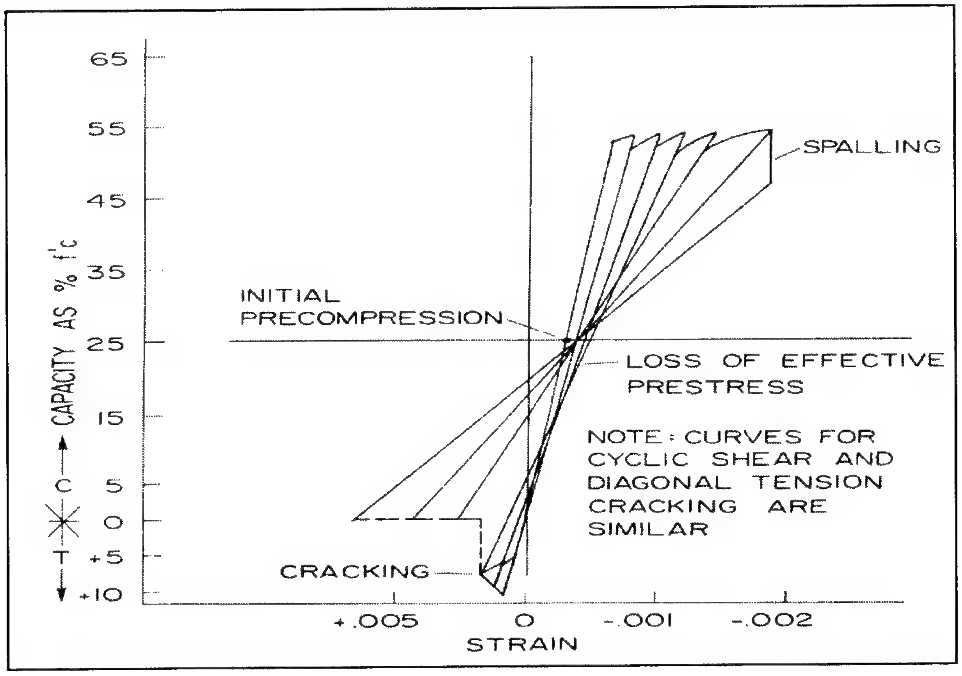


Figure 37. Schematic of vertical prestressing tendons in wall of concrete offshore platforms subjected to cyclic membrane shear

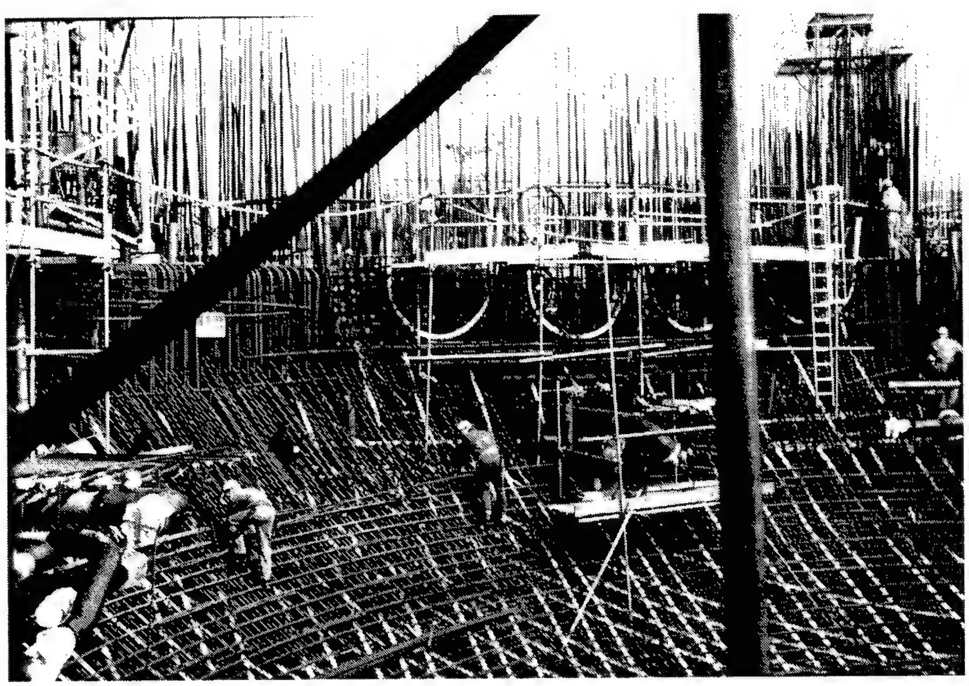


Figure 38. Laying out post-tensioning ducts in offshore platform "troll"

A case study: failure of Lacey V. Murrow Bridge

A typical example of fatigue failure of a concrete marine structure is the catastrophic failure of the Lacey V. Murrow Floating Bridge across Lake Washington near Seattle. The 2,012-m-long bridge—the world's first major floating concrete bridge—was built in 1939. It consisted of 22 pontoons bolted together to form a continuous floating structure. Each pontoon was about 105 ft long, 18 m wide and 4.5 ft deep and weighed about 5,000 tons.

At the time of the failure, the structure was undergoing a major renovation designed to widen the two-lane deck to accommodate an additional breakdown lane. The construction was suspended on Wednesday, November 21, 1990, for the Thanksgiving holiday weekend.

On November 22-23, a very strong and usually lengthy storm struck the site, generating substantial wind and wave loads on the bridge. The storm was followed by heavy rains of over 4 in. (100 mm) on November 23-24. Early on Sunday, November 25, a pontoon near the center of the bridge (pontoon A5) gradually sank into the water and eventually broke in half in the middle, dragging an additional seven pontoons down about 60 m to the bottom of the lake. Figure 39 shows the remaining portion of the bridge after the sinking of the middle pontoons. The surviving pontoons, as well as the debris on the lake bottom, were moved to a temporary site on the lake for inspection.

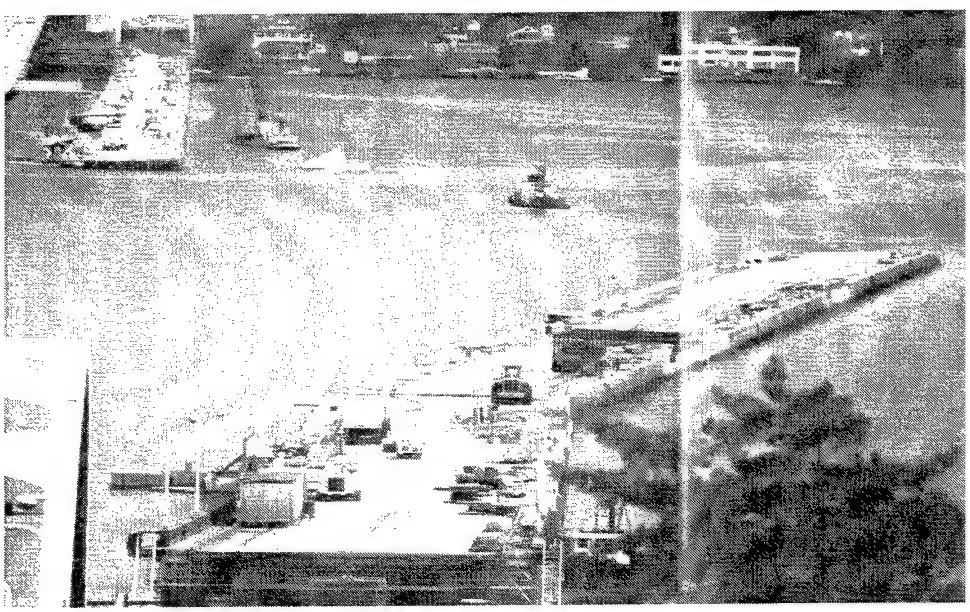


Figure 39. Surviving bridge after sinking of the eight pontoons

Following the incident, a thorough investigation into the sudden collapse of the bridge was conducted (Ben C. Gerwick, Inc. 1991b). The investigation concluded that many factors contributed to the collapse of the bridge; however, the main cause of the failure was fatigue degradation of the bond strength at the lap splice of bottom reinforcing steel of the pontoon. On the basis of forensic evidence, the

investigation reconstructed a probable sequence of events leading to final collapse of the bridge.

First, it should be noted that the 1930s design of the Lacey V. Murrow Bridge reflects the limitations of the state of engineering practice at the time. Specific defects in the design which contributed to the fatigue failure 50 years later included these:

a. Fatigue limit state was not considered in the design, nor was there any recognition of the adverse effects of submerging concrete and cycling between the tension/compression stress range.

b. The principal reinforcement in the bottom slab consisted of 1-1/8 in. (28-mm-) square bars of poor bond characteristics.

c. Fifty percent of the longitudinal bars were spliced at a single location. Such concentration of splices caused high stress concentration, which led to cracking, loss of the bond, and progressive bond slip.

d. The overlap length of the lap splices was inadequate by the modern design codes.

e. There was no through-slab confinement reinforcement at the splices, as required today for splices in axial tension.

f. Torsional shear was neither well understood nor a design consideration in the 1930s design. As a result, the reinforcement splices at the corners of the deck were inadequate to develop the full ultimate torsion capacity of the pontoon under major storms.

g. There was minimal transverse steel confinement in the pontoons.

The design was not based upon a dynamic analysis, but on a quasi-static beam-on-elastic-foundation analysis method. There were very low still-water bending moments in the pontoons, because the distribution of the dead load is closely matched by the uniform distribution of the buoyancy. Consequently, even moderate wave loading would create stress reversals in the top and bottom slabs. Dynamic wind and wave loads on the floating bridge constitute over 90 percent of the design loads. As a comparison, for a typical highway bridge, dynamic loads due to live load range from 6 to 20 percent of the total design load.

During its 50-year service life, the bridge was subjected to many severe storms. Based upon the wind records and wave hindcasting studies, it is estimated that there had been over 16 millions of wave of sufficient height to cause significant cycles of bending and torsion. Although individual waves were not high enough to cause cracking in the pontoons, the cumulative effects of cycling excursion into the tension stress range of the concrete did cause numerous cracks on the bottom slab and slides of the pontoons.

For the first 15 years, the bridge had been generally crack-free. Since the first major crack in the pontoon A5 was reported in 1960, more and wider cracks developed each winter. By 1990, extensive transverse cracks were observed in the top and bottom slabs, and on the side walls of the pontoons. Figure 40 shows the transverse cracks in the bottom slab of one of the pontoons. These cracks were due to fatigue degradation, aggravated by the submergence of the concrete in water. The pumping and wedging actions of the water in and out of cracks exacerbate the crack widening and detrimental leaching.

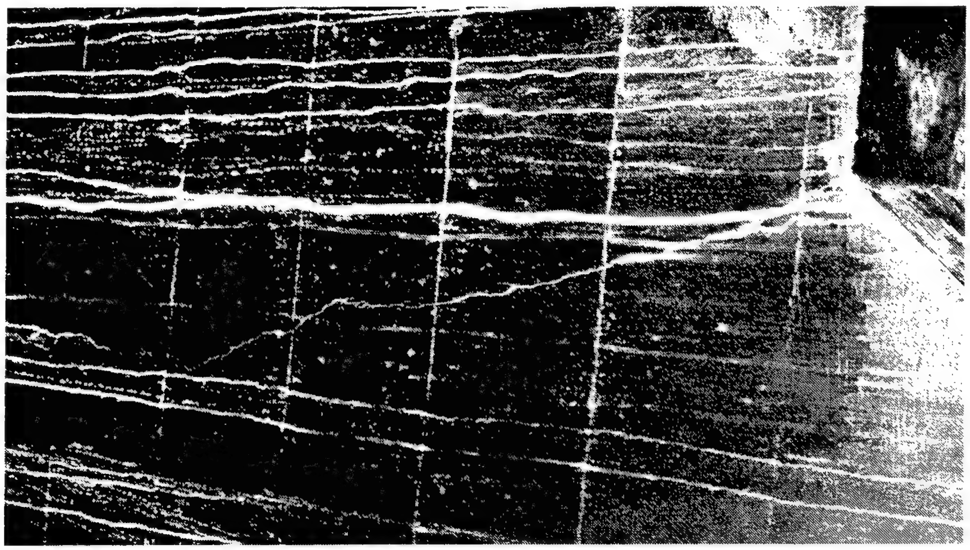


Figure 40. Transverse cracks in the bottom slab of the broken pontoons

The cyclic bending and torsion were accompanied by lateral and vertical shear forces, resulting in substantial cracking on the side walls. The subsequent opening and closing of these cracks led to cumulative bond degradation of the main longitudinal reinforcement. The bond degradation consisted of loss of adhesion and micro/macro internal cracks due to splitting along the bars that were not adequately confined and had very poor bond characteristics. As a result, progressive bond slip occurred. The cracks did not fully close after each storm and become even wider during the next storm. Prior to the 1990 renovation of the bridge, almost all the cracks seeped, and some were visibly leaking.

The fatigue failure of the bond is clearly demonstrated by the forensic evidence, as shown in Figures 41 and 42. Figure 41 shows long lengths of bare steel bars projecting from the broken concrete, without any adhering of concrete. Figure 42 shows the complete debonding of reinforcing steel and concrete in the bottom slabs.

During the storm of November 22-23, the heavy rains and sprays from the lake partially filled the pontoon, mainly in the cells of the north side, creating a considerable amount of bending and torsional moments on the pontoon. The condition of the pontoon prior to the failure is illustrated in Figure 43. The pontoon, already weakened by prior cracks, was excited to relatively high dynamic stresses in tension and torsional shear by the waves.

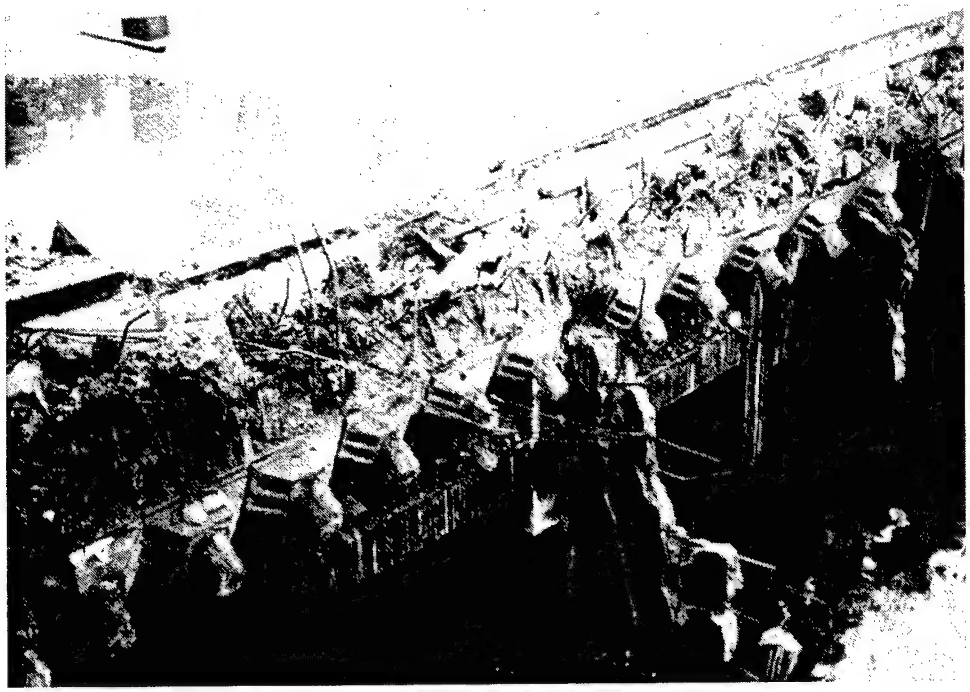


Figure 41. Broken end of surviving pontoon (long length of bare steel bars projecting from broken concrete indicates loss of bond)

Figure 42. Bare steel bars in bottom slab of broken pontoons

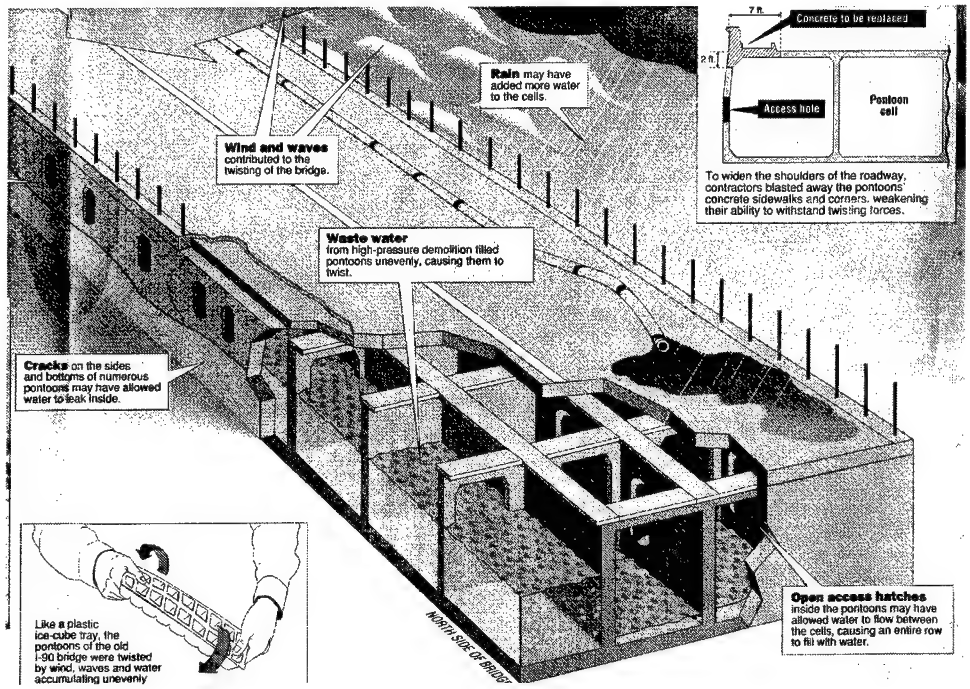


Figure 43. An illustration of the pontoon's condition prior to the sinking

The torsional shear capacity of the pontoon was substantially reduced by the transverse cracks in the pontoon, the cutout side openings for the renovation, and the membrane tension at the bottom slab due to flexural bending. As a result, diagonal shear cracking was developed in the pontoon at the height of the storm the Friday night before the failure. Such shear cracks tend to stay open even after the torsion is removed, because of the wedging action of coarse aggregates along the crack surfaces and accumulated bond slip.

The permanent cracks caused water inflow to increase at a progressively increasing rate. The in-leakage was slow at the beginning. As more water accumulated in the pontoons, the vertical bending moments increased and further widened the cracks in the bottom slabs, which in turn accelerated the leakage. Calculation of the inflow rate and structural analysis show that sufficient water would have leaked into the pontoon A5 by about 8:00 a.m. on November 25 to cause it to sink. The final failure mode of the pontoons was in bending plus torsion, with the longitudinal reinforcement splices splitting out in bond failure.

It should be pointed out that the Lacey V. Murrow Bridge was reinforced with passive reinforcement only. All the other floating bridges over Lake Washington, which were built at later times, used prestressing technologies. However, even some of the prestressed concrete floating bridges (such as the Old Hood Canal Bridge and the Evergreen Bridge) have experienced significant cracking and leaking in their pontoons under cyclic loading. Floating structures are frequently subjected to bending, shear, and torsion under repeated loading. At present, fatigue behavior and cumulative damage of prestressed concrete floating structures have not been well understood. Further study into the effects of prestressing and proper fatigue design criteria is highly recommended.

5 Conclusions

Since its first marine application in piling in the 1950s, prestressing technology has essentially changed the ways of designing and constructing offshore structures. Various innovative developments include pier and wharf structures, offshore gravity-based platforms, arctic structures, floating oil/gas platforms, floating bridges, floating marine terminals, and dry docks. Primary reasons for the wide acceptance of prestressing technology in offshore applications include its constructibility, durability, and overall economy. If properly designed and constructed, prestressed concrete has proven to enhance the strength and serviceability of offshore structures, especially in shear resistance, fatigue resistance, crack control, and watertightness.

The single most important factor underlying the durability of offshore concrete structures is permeability of the concrete. Based on past experience, the offshore construction industry has established "ten golden rules for durable concrete in the sea," which cover stringent requirements on materials selections, concrete production, construction planning and requirements, and durability design.

Past experience shows that the majority of prestressed concrete offshore structures have exhibited remarkably sound and reliable behavior in hostile marine environments. Typical damages in offshore structures have been impacts from ships and dropped objects, or leakage due to bad construction joints. Fatigue resistance can become a major concern, if the structures are not prestressed or were not designed in accordance with modern design standards.

The design criteria for offshore gravity-based structures include load criteria, Ultimate Limit State, Serviceability Limit State, Fatigue Limit State, and Progressive Collapse Limit State. Furthermore, the detailing and construction requirements are essential parts of the design requirements.

The construction of a typical offshore platform involves many complex load stages. The critical loads on the structure vary greatly in magnitude, location, direction, and duration from one load stage to another. The load criteria must address all the critical load combinations in each load stage. In the past, most errors incurred have been due either to overlooking an intermediate substage or due to combining two or more substages in order to save computational efforts.

The purpose of specifying crack control criteria is to meet some special performance requirements of offshore structures. Due consideration must be given

to relevant design parameters, such as fatigue strength provision, concrete cover, and quality. The importance of the minimum reinforcement requirement for crack control cannot be over emphasized.

The crack control design may adopt either allowable stress criteria or crack width criteria. The allowable stress criteria are an empirical approach. Their adequacy for crack control has been demonstrated by the excellent performance of early structures in severe environments.

Prediction of the crack width is a very approximate process. Nevertheless, the calculation of crack width can have a significant impact on the resulting design. In the past, use of the crack width criteria has not always resulted in cost-effective design, because the computer analyses often pinpoint unrealistic peak stress values as infringements of design criteria that have no significant consequence to the serviceability and durability. In some cases, the reinforcement requirement to control cracking can run up to 30 percent higher than that required for satisfying the ultimate strength requirements. Therefore, the allowable stress criteria are generally recommended for crack control design. The crack width criteria should be applied only in special areas, such as the splash zone, critical structural members susceptible to fatigue damage, or structural members that do not meet the allowable stress criteria.

Offshore design codes and specifications often accept three design methodologies for checking the ULS requirements:

a. The simplified equilibrium approach on the basis of the principle of superposition of component resistance.

b. The multi-axial stress field method on the basis of Modified Compression Field Theory.

c. Variable strut-tie model.

For membrane elements, the traditional equilibrium design method is generally acceptable for practical design. For shell subjected to combinations of membrane forces and bending, the flexural strength of the elements is strongly influenced by the membrane shear force. The interaction between membrane forces and bending must be properly included in design.

When shell elements are subjected to transverse shear, membrane loads, and bending, it is important that all the loads be properly considered in shear capacity calculation. If the membrane loads and bending are moderate, the traditional design method on the basis of equivalent beam approximation and the principle of superposition has proven to be generally satisfactory. However, when high bending moments and high axial forces occur simultaneously with transverse shear, the equivalent beam method will lead to inaccurate results. In such cases, more refined analysis methods, such as the multi-axial stress field method, should be used to evaluate the sectional resistance.

Confinement is an important detailing requirement in offshore platforms. All the prestressing tendons should be confined between layers of transverse reinforcing steel in slabs and walls. Curved prestressing tendons should be adequately confined by lateral reinforcement.

Fatigue resistance is an important design consideration. In general, high-cyclic fatigue load has not been considered as a significant problem. However, significant damage can occur at the low-cycle, high-amplitude end of the load spectrum. That is, a relatively small number of load cycles of high magnitude can cause a sizable reduction in stiffness and rapid increases in strains that lead to cracking and spalling.

There is a significant difference in fatigue endurance between uncracked prestressed concrete and concrete that has been precracked by a prior extreme load. Subsequent to cracking, repeated opening and closing of the crack will lead to significant stress increases in steel, in concrete (due to hammering), degradation of bond strength, and loss of prestress. These detrimental effects are especially pronounced for submerged concrete due to hydraulic fracturing and erosion of the cement hydrates. The current design practice for marine structures is to prestress concrete so that the concrete stresses rarely cycle into the tension range.

Fatigue design of offshore concrete structures can be conducted based on either stress limit control or comprehensive fatigue analysis. The stress limit control is generally recommended for practical design. The comprehensive fatigue analysis should be based on cumulative damage and should include bond, flexural, and shear strength. The comprehensive fatigue analysis is usually too complex for practical design and should be limited to special structures or structural components for which fatigue is likely to be a serious problem.

The Progressive Collapse Limit State corresponds to the condition that failure of one member due to accidental or abnormal overloads leads to progressive failure of adjoining members, and eventually to collapse of the structure. Design against progressive failure is usually achieved with structural strength and ductility. Ductility should be defined both at the structural element level and the structural system level.

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Appendix A Recommendations for Further Research

Prestressed concrete historically has had very limited use in inland waterway navigation structures. Recent development of the in-the-wet construction method potentially offers a major opportunity for innovative applications of prestressed concrete in navigation structures.

Although in-the-wet construction shares many similarities with the offshore construction discussed in this report, offshore experience and design standards may not be directly applicable for inland waterways without due consideration of their special restrictions and requirements. In this regard, properly directed research efforts in some key technical areas are essential to successful implementation of prestressing technology. This appendix is a list of special topics of prestressing technology that deserve further investigation for application to navigation structures.

Guide Specifications and Standard Practice Codes

Precast/prestressed concrete historically has had very limited use in inland waterway navigation structures. With recent developments in innovative construction, precast/prestressed concrete will potentially become one of the most common and important types of structural components in hydraulic structures.

Ensuring the quality and production efficiency of precast/prestressed concrete segments will be one of the challenges that U.S. Army Corps of Engineers will face during implementation of the new construction method. It is now critical for the Corps to establish an up-to-date standard for designing and specifying precast/prestressed concrete. It is also important to provide field engineers and contractors with adequate and simple-to-follow guidance for precast/prestressed concrete construction and inspection, including acceptable product standards and quality assurance/quality control plans.

Lightweight Concrete

The design and construction of prefabricated concrete modules is frequently restricted by various project-specific constraints. Often, the draft requirement for float-in segments necessitates that the segment be outfitted in deep water and that the access route to the site be extensively dredged. It is also likely that heavy lift-in segments would force the contractor to purchase new lift equipment. Therefore, a 10- to 15-percent weight reduction in the precast modules will lead to significant improvements in design and construction, saving tens of million dollars. In this regard, using precast modules made of lightweight concrete represents an important potential application for innovative construction.

Many outstanding examples exist of using high-quality lightweight concrete in large-scale marine structures. However, a number of lightweight concrete structures have also experienced durability problems, due largely to improper lightweight concrete mixtures and improper construction quality control. Thus, a review of the state-of-the-art lightweight technology is important for implementing successful applications.

Serviceability Requirements of Hydraulic Structures

The Corps of Engineers has traditionally used a hydraulic factor of 1.3 in the ultimate strength design to cover the serviceability requirements. This methodology may serve well for conventional hydraulic structures that contain lightly reinforced or unreinforced low-strength concrete (commonly 3,000 psi, 21 MPa). With the in-the-wet construction method, hydraulic structures are constructed with high-strength, high-quality precast/prestressed concrete shells that are heavily reinforced and later filled with tremie concrete. For this new type of hydraulic structures, the design methodology embodied in the hydraulic factor is likely to be overly conservative.

Past experience with offshore structures indicates that overemphasis on the serviceability often results in uneconomical design and constructibility problems due to congestion of reinforcement. Therefore, an in-depth investigation into the relationship between the serviceability and design load factors is a pressing need. This investigation should cover all the critical design parameters (concrete cover, concrete strength and quality, area and distribution of reinforcement, effects of prestressing, etc.) as well as the serviceability requirements including (abrasion resistance, crack width requirement, fatigue strength, resistance to freeze-thaw damage, impact absorption, and permeability).

Design Methods for Shells and Plates

The current design for shells and plates subjected to combinations of high in-plane shear, membrane forces, and bending moments adopts the ACI code, which

makes the equivalent beam assumption. This approximate method works well for building structures in which high transverse shear on the components is not combined with high in-plane shear and membrane forces. For in-the-wet construction, float-in segments, lift-in segments, and permanently floating structures will be subjected to more complex load combinations. Studies show that the simple equivalent beam approach may lead to inaccurate design. Therefore, a rational approach based upon triaxial stress-strain relations (Modified Compression Field Theory) is recommended to develop a consistent and practical design criteria.

Applications of Posttensioning

For in-the-wet construction, using large precast concrete segments generally has significant advantages over smaller segments, including minimum underwater connections, a minimum weather-dependent work window, minimum risk factor associated with positioning and setting of the precast segments, and minimum shutdown time of any adjacent existing structures for construction. However, the size of float-in segments is limited in their length-to-depth ratios in order to keep longitudinal stress to an acceptable level during transport and immersion.

Prestressing plays a critical role in determining the optimum size and shape of precast segments. Unfortunately, the numerous previous feasibility studies of in-the-wet construction have not focused on prestressed concrete modules (USACE 1993, 1996; Ben C. Gerwick, Inc. 1994, 1997; Bergmann Associates et al. 1997).

At present, the advantages and practicality of large-scale prestressed concrete float-in modules are not fully understood. Therefore, a systematic evaluation of prestressed/precast concrete modules is currently of significant value in improving the design practice. The study should identify the types of precast concrete modules and determine the nature of loads on them. Then, conceptual design should be conducted to explore the potential advantages of prestressing. Standard detailings and typical tendon profiles for common concrete modules should be developed. Special marine techniques, such as overwater joining of precast segments and ballasting sequence to set down the module, are also important.

Assessment of Fatigue Strength of Floating Structures

Floating guide walls are likely to develop cracking under repeated barge impact. Repeated barge impact is a type of high-amplitude, low-cycle loading that has been found to be primarily responsible for fatigue degradation of concrete structures. Barge impact on the guide walls is perceived to be a much more severe load than traffic loads over floating bridges or storm waves on offshore platforms.

At present, fatigue behavior and the cumulative damage of prestressed concrete floating structures are not well understood. An investigation of fatigue behavior of floating navigation structures is of significant value. It is recognized that

comprehensive fatigue analyses are generally too complex for practical design purposes. The investigation should first establish a practical load criterion for barge impact loading. Then, the study should develop reliable but simple fatigue design criteria and design guidelines for floating structures under repeated impact loads.

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This report presents a review of the current design practice in offshore prestressed concrete structures. Major topics include recent applications of prestressing technology in offshore concrete gravity-based structures, current design criteria, and durability design requirements. An appendix summarizes recommendations for further research that is needed to effectively apply prestressing technology to inland waterway navigation structures.

Conclusions regarding prestressing technology are that it has gained wide acceptance due to its benefits in the areas of constructibility, durability, and overall economy. If properly designed and constructed, prestressed concrete has proven to enhance the strength and serviceability of offshore structures, especially with regard to shear resistance, fatigue resistance, crack control, and watertightness. The majority of prestressed concrete offshore structures have exhibited remarkably sound and reliable behavior in hostile marine environments.

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